CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

DECEMBER 1959

VOL. LIV. NO. 12

FIFTY-FOURTH YEAR OF PUBLICATION

PRICE 2s. ANNUAL SUBSCRIPTION 24s. POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 625 ISSUED MONTHLY Registered for Canadian Magazine Post

INDEX OF ADVERTISERS

1959.

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LIV, No. 12. LONDON, DECEMBER, 1959.

Increasing Productivity.

It is commonly said that when labour was cheap there was no harm in using it extravagantly, and that the need to save labour becomes desirable or necessary only as its cost increases. This argument fails to recognise that the words cheap and dear have no meaning unless they are related to other prices. When a man was paid a shilling an hour and could provide a dinner for his family for eight pence, and his employer paid the then price for a motor-car, the cost of labour and goods was relatively the same as it is when a man is paid six shillings an hour and can provide a dinner for his family for four shillings, and the employer pays six times as much for a car. Then, as now, builders built for profit and endeavoured to get the best value for the money they paid in wages. The difference is that men now work fewer hours a year and builders need more profit so that a reasonable reward will remain after taxation, and all need more to keep pace with inflation. The result is that employers have to think harder, to devise new ways of getting more production by the use of machinery, to eliminate unnecessary work, to avoid men being idle through no fault of their own, and to pay bonuses to encourage men to work harder. Some of these means of increasing production, particularly payment by piecework, have been in use for centuries. The application of all these means has now been intensified.

This is only partly shown in a bulletin entitled "Better Methods in the Building Trades" * which purports to describe methods of reducing costs adopted by eight builders, one sub-contractor, and one builders' merchant. In these cases work study departments have been set up or work study specialists employed, and their recommendations have resulted in increasing production and in saving cost. Much of the twenty pages of text in the bulletin is devoted to laudatory remarks on the merits of work study and on the firms whose enterprise is described. There are also some examples of the methods by which greater production, and consequently lower costs, have been achieved. These are all so obvious that the only impression one gets in reading about them is of amazement that the state of affairs that has been remedied could have been allowed to exist. The praise that is lavished upon these firms for seeing that their men did not waste their time in such ridiculous ways reminds one of the different treatment meted out by a noble lord on hearing that a burglar had got into his castle: he sent for the butler and gave him a golden sovereign for catching the burglar and a week's

* Published by the British Productivity Council. Price 5s.

notice for allowing him to get in. The stupid ways in which men were found to be wasting time and materials are indicated in the following examples given in the bulletin.

Instead of erecting a working platform under the whole of a factory ceiling for the use of painters and electricians, platforms were erected for two bays only so that while men were working on one platform the other was being dismantled and re-erected. Production was increased by one-fifth by ensuring that bricklayers always had a supply of bricks and mortar on the scaffold. In a case where floor slabs of mixed sizes were delivered by lorry, the men unloaded the slabs much quicker than they were laid because the paviors had to sort out the slabs; the slabs were laid twice as fast when the unloaders, instead of standing idle, arranged the slabs in the order in which they were needed. By using a ladder instead of a staging two men were able to fix small plastic wall panels at a height of 30 ft. at the rate of three minutes each instead of twenty. A " work study engineer " discovered that the marking of glazing-bars could save much time on the site. Six carpenters fixing joinery queued to use one drill; when this was discovered three drills were supplied. On the same works skirting boards of various lengths were stored together and sorted by the carpenters—" those who were painstaking enough to search for the right lengths lost time; the others had to waste material cutting down excessively large pieces to size "—time and material were saved by storing the various sizes separately. A " work study man " pointed out that time could be saved if carpenters' benches were nearer to their work.

In a precast concrete shop a team of six men included two carpenters who generally worked part time only, and the work was arranged so that they could also do other work. Much time was spent in making and using small pieces of shuttering to form grooves at the tops of columns to receive the ends of precast beams, and in grouting these connections; a saving of 15 per cent. in the cost of shuttering was made by forming slots in the top of the column shutters to receive the ends of the beams, supporting the beams in these slots, and then concreting the columns. A saving was made in the erection of a multiple-story building by leaving a hole in each floor through which the centering and its supports could be passed for use on the floor above, instead of passing the material through doors and windows and hoisting it up outside the structure. Savings were made by using baseboard instead of plaster-board, and by supplying and fixing plasterboard instead of having the work done by a sub-contractor.

In all the cases mentioned in this bulletin it was found that the men co-operated willingly when it was explained to them that increased production would result in increased earnings, but it is not stated why the simple and obvious improvements could not have been made by the builder's own staff and without the extra cost of works study specialists and in some cases " site meetings every fortnight with military regularity ". Indeed the bulletin will probably be most welcomed by those who contend that the building industry is so inefficient that it should be nationalised. It is tragic that this publication should be issued at a time when a really useful account could be written of the recent achievements of the building industry in reducing cost and construction time by the use of new plant, new processes, and new materials, in which it is doubtful if work study specialists had any part.

Design of Chimneys for Resistance to Weight and Wind.

By LESLIE TURNER, B.Sc., M.I.C.E., PP.I.Struct.E.

As in the case of most structures, the design of reinforced concrete chimneys is a matter of trial and error. The calculations are complicated further because the effects of differences and changes of temperature have to be considered and combined with those due to the weight of the chimney and the effect of wind. In the following a method using charts is described whereby the stresses due to weight and wind can be readily determined. The combination of these stresses with the stresses due to temperature has been described elsewhere.*

The height and internal dimensions of a chimney are determined from considerations of thermodynamics, experience, and probably the requirements of national and local authorities, and will be given to the designer by those responsible for the plant and process from which the chimney is required to conduct gases. The maximum temperature and chemical analyses of the gases entering at the flue must therefore be known. From these data the material of construction, the thickness, the method of construction, and the protection of the inner face of the concrete must first be determined. In the past some concrete chimneys have not been lined, or lined for only part of their height. This may be satisfactory for cool and inert gases, such as warm clean air, but it is not recommended for exhausts from boilers of any kind and certainly not from industrial plants.

In determining the thickness of the shaft it must be borne in mind that it must remain intact after periodical heating and cooling when it is out of use. A final consideration is whether the lining shall be free-standing, that is whether its whole weight shall be carried on its lowest course or whether it shall be supported at intervals throughout its height on corbels or brackets built monolithically with the shaft. In this case the thickness of the lining will be less at the bottom, but its weight will be carried by the concrete shaft.

When the height of the chimney is known, the outside diameter at any horizontal plane depends on the internal diameter required, the air-space (if any) between the shaft and the lining, and the thickness of the lining. A slight taper is desirable for the sake of the appearance of the chimney, but it may not be essential.

A tentative elevation should be drawn and divided into sections of height, usually about 20 ft. to 25 ft. in the case of tall chimneys, and the weight of the shaft, including any lining carried by that part of the shaft and the corbels on which it is supported, should be evaluated at each of these horizontal sections. Bending moments due to the wind at these sections must also be calculated, the average wind pressure on every trapezoidal area being multiplied by the distance from the horizontal section to the centroids of the areas above the section. Having determined the weight carried on each horizontal section and the bending moment due to the wind at each section, other requirements, such as the thickness of the shaft and the percentage of reinforcement, are assumed if they are not known. The stresses due to wind and weight are then calculated. If the difference in temperature through the wall of the shaft is considerable it may be

* "Reinforced Concrete Chimneys." By C. P. Taylor and Leslie Turner (Concrete Publications, Ltd.).

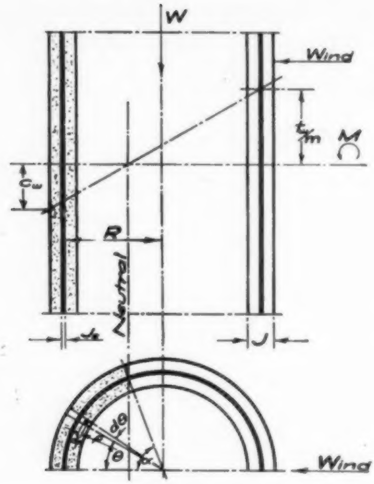


Fig. 1.

necessary for the maximum stress due to wind and weight to be not more than about half the compressive stress permissible in the concrete. The corresponding tensile stress in the reinforcement should be about one-tenth or less of the permissible stress in order that the final stresses, taking into account those due to temperature, do not exceed the permissible stresses. It is also inherent in the nature of the problem that the tensile stresses in the reinforcement cannot approach the ordinary working stresses, although some specifications quote such limiting stresses.

Analysis of Annular Sections.

The assumptions made in the analysis which follows are those for the ordinary elastic theory of reinforced concrete.

NOTATION.

c_w, maximum compressive stress in concrete due to wind and weight only (lb. per square inch).

J, thickness of shaft (in.).

J_s, thickness of an imaginary steel shell of mean radius R having a cross-sectional area equal to the actual area of the vertical reinforcement bars (in.).

M, moment of the wind about the section (in.-lb.).

m, modular ratio.

mc_w, maximum compressive stress in the vertical reinforcement due to wind and weight only (lb. per square inch).

p, percentage of vertical reinforcement $= \frac{100J_s}{J}$; r = proportion of vertical reinforcement $= \frac{J_s}{J} = \frac{p}{100}$: total area of reinforcement $= 2\pi RJ_s$ (sq. in.).

R, mean radius of shaft (in.)
t_w, maximum tensile stress in the vertical reinforcement due to wind and weight only (lb. per square inch).
W, weight carried on the horizontal section (lb.).
ρ, thickness of element of shaft.
θ, angle defining position of element of shaft.
α, half angle subtended by compressive zone of shaft.
e, eccentricity $\left(=\dfrac{M}{W}\right)$ at any horizontal section.

BASIC FORMULÆ.—The analysis of an annular section proceeds as in the following.

Consider element $\rho.d\rho.d\theta$ of concrete shell.

Stress in element $$= \frac{\rho\cos\theta - R\cos\alpha}{R+\frac{t}{2} - R\cos\alpha}\, c_w.$$

Total compression in element $$= \frac{\rho\cos\theta - R\cos\alpha}{R+\frac{t}{2} - R\cos\alpha}\, c_w\, \rho.d\rho.d\theta.$$

Moment of element about N.A. $$= \frac{(\rho\cos\theta - R\cos\alpha)^2}{R+\frac{t}{2} - R\cos\alpha}\, c_w\, \rho.d\rho.d\theta.$$

∴ Integrating from $\rho = R-\frac{t}{2}$ to $\rho = R+\frac{t}{2}$,

total compression in element $d\theta$ $$= \frac{c_w.d\theta}{R+\frac{t}{2}-R\cos\alpha}\int_{R-\frac{t}{2}}^{R+\frac{t}{2}}\left(\rho^2\cos\theta - \rho R\cos\alpha\right)d\rho$$

$$= \frac{c_w.d\theta}{R+\frac{t}{2}-R\cos\alpha}\left[\left(R^2t+\frac{t^3}{12}\right)\cos\theta - R^2t\cos\alpha\right].$$

Total moment of element $d\theta$ about N.A. $$= \frac{c_w.d\theta}{R+\frac{t}{2}-R\cos\alpha}\int_{R-\frac{t}{2}}^{R+\frac{t}{2}}\rho\left(\rho\cos\theta - R\cos\alpha\right)^2 d\rho\,.$$

$$= \frac{c_w.d\theta}{R+\frac{t}{2}-R\cos\alpha}\left[\left(R^3t+\frac{Rt^3}{4}\right)\cos^2\theta - \left(2R^3t+\frac{Rt^3}{6}\right)\cos\theta\cos\alpha + R^3t\cos^2\alpha\right].$$

Stress in steel at θ $$= m.c_w\,\frac{R\cos\theta - R\cos\alpha}{R+\frac{t}{2}-R\cos\alpha}\,.$$

∴ Total force in steel in element $d\theta$ $$= \frac{t\,r\,m\,c_w}{R+\frac{t}{2}-R\cos\alpha}\left(R\cos\theta - R\cos\alpha\right)R.d\theta.$$

∴ Moment of this force about N.A. $$= \frac{t\,r\,m\,c_w}{R+\frac{t}{2}-R\cos\alpha}\left(R\cos\theta - R\cos\alpha\right)^2 R\,d\theta.$$

Integrating for concrete between 0 & α and for steel between 0 and π,
Total net compression in half shell

$$= \frac{c_w}{R+\frac{t}{2}-R\cos\alpha}\int_0^{\alpha}\left[\left(R^2t+\frac{t^3}{12}\right)\cos\theta - R^2t\cos\alpha\right]d\theta + \frac{R^2t\,r\,m\,c_w}{R+\frac{t}{2}-R\cos\alpha}\int_0^{\pi}\left(\cos\theta-\cos\alpha\right)d\theta$$

$$= \frac{c_w}{R+\frac{t}{2}-R\cos\alpha}\left\{\left(R^2t+\frac{t^3}{12}\right)\sin\alpha - R^2t\alpha\cos\alpha - R^2t\,r\,m\,\pi\cos\alpha\right\}.$$

Similarly total moment of internal forces in half shell about N A

$$= \frac{c_w}{R+\frac{J}{2}-R\cos\alpha}\int_0^\alpha\left[\left(R^3J+\frac{RJ^3}{4}\right)\cos^2\theta-\left(2R^3J+\frac{RJ^3}{6}\right)\cos\theta\cos\alpha+R^3J\cos^2\alpha\right]d\theta$$

$$+\frac{Jrmc_w R^3}{R+\frac{J}{2}-R\cos\alpha}\int_0^\pi(\cos\theta-\cos\alpha)^2\,d\theta$$

$$= \frac{c_w}{R+\frac{J}{2}-R\cos\alpha}\left[\left(R^3J+\frac{RJ^3}{4}\right)\left(\frac{\alpha+\frac{1}{2}\sin 2\alpha}{2}\right)-\left(2R^3J+\frac{RJ^3}{6}\right)\sin\alpha\cos\alpha+R^3J\alpha\cos^2\alpha\right.$$

$$\left.+\,JrmR^3\left(\frac{\pi}{2}+\pi\cos^2\alpha\right)\right].$$

Let W = *applied direct load;* M = *applied moment about centre line,* $e=\frac{M}{W}$

Then applied moment about N.A $= M-WR\cos\alpha$.

$$W = \frac{2c_w}{R+\frac{J}{2}-R\cos\alpha}\left\{\left(R^2J+\frac{J^3}{12}\right)\sin\alpha-R^2J\alpha\cos\alpha-R^2Jrm\pi\cos\alpha\right\}$$

$$\text{or } W = \frac{2c_wRJ}{1+\frac{J}{2R}-\cos\alpha}\left\{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha\right\}.$$

$$\text{Also } M-WR\cos\alpha = \frac{2c_wR^2J}{1+\frac{J}{2R}-\cos\alpha}\left[\left(1+\frac{J^2}{4R^2}\right)\left(\frac{\alpha+\frac{1}{2}\sin 2\alpha}{2}\right)-\left(2+\frac{J^2}{6R^2}\right)\sin\alpha\cos\alpha\right.$$

$$\left.+\,\alpha\cos^2\alpha+rm\left(\frac{\pi}{2}+\pi\cos^2\alpha\right)\right].$$

$$\frac{M-WR\cos\alpha}{W} = \frac{R\left[\left(1+\frac{J^2}{4R^2}\right)\left(\frac{\alpha+\frac{1}{2}\sin 2\alpha}{2}\right)-\left(2+\frac{J^2}{6R^2}\right)\sin\alpha\cos\alpha+\alpha\cos^2\alpha+rm\left(\frac{\pi}{2}+\pi\cos^2\alpha\right)\right]}{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha}$$

$$= \frac{R\left[\left(1+\frac{J^2}{4R^2}\right)\frac{\alpha}{2}-\left(\frac{3}{2}+\frac{J^2}{24R^2}\right)\sin\alpha\cos\alpha+\alpha\cos^2\alpha+rm\left(\frac{\pi}{2}+\pi\cos^2\alpha\right)\right]}{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha}.$$

$$\text{Then } \frac{e}{R}-\cos\alpha = \frac{\left(1+\frac{J^2}{4R^2}\right)\frac{\alpha}{2}-\left(\frac{3}{2}+\frac{J^2}{24R^2}\right)\sin\alpha\cos\alpha+\alpha\cos^2\alpha+rm\left(\frac{\pi}{2}+\pi\cos^2\alpha\right)}{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha}.$$

$$\therefore \frac{e}{R} = \frac{\left(1+\frac{J^2}{4R^2}\right)\frac{\alpha}{2}-\left(1-\frac{J^2}{12R^2}\right)\frac{\sin\alpha.\cos\alpha}{2}+rm\frac{\pi}{2}}{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha}.$$

$$\text{Also } \frac{W}{RJ} = \frac{2c_w}{1+\frac{J}{2R}-\cos\alpha}\left\{\left(1+\frac{J^2}{12R^2}\right)\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha\right\}.$$

For all practical purposes higher powers of $\frac{J}{R}$ *may be neglected*

$$\text{Then } \frac{e}{R} = \frac{\alpha-\sin\alpha.\cos\alpha+rm\pi}{2(\sin\alpha-\alpha\cos\alpha-rm\pi\cos\alpha)} \quad \text{---------} \quad (1)$$

$$\text{and } c_w = \frac{W}{RJ}\times\frac{1+\frac{J}{2R}-\cos\alpha}{2(\sin\alpha-\alpha.\cos\alpha-rm\pi\cos\alpha)} = \frac{W}{RJ}\times K_w \quad \text{—}(2)$$

Also, if t_w = *max. tensile steel stress,*

$$\frac{t_w}{mc_w} = \frac{R+R\cos\alpha}{R+\frac{J}{2}-R\cos\alpha} \text{ or } \frac{t_w}{c_w} = \frac{m(1+\cos\alpha)}{1+\frac{J}{2R}-\cos\alpha} \quad \text{——}(3)$$

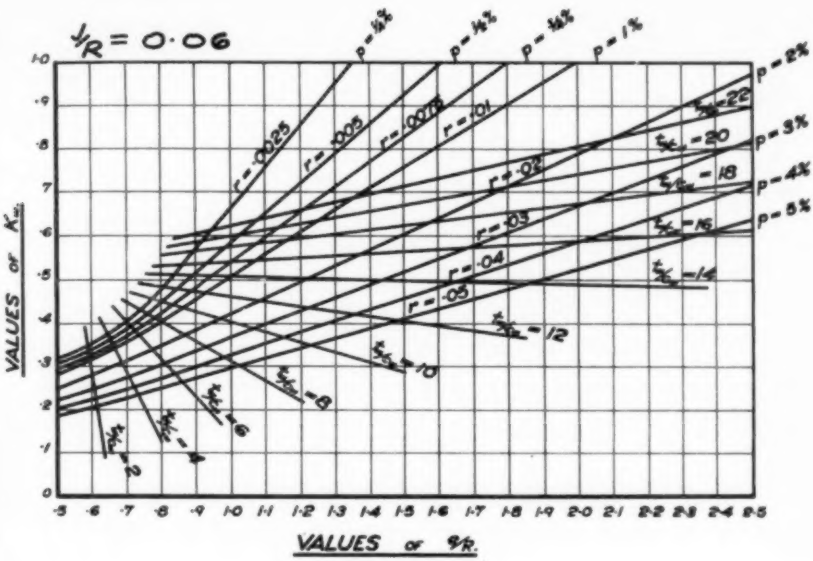


Fig. 2.

The three essential formulæ in the foregoing are:

Eccentricity ratio: $\dfrac{e}{R} = \dfrac{\alpha - \sin\alpha\cos\alpha + \pi rm}{2(\sin\alpha - \alpha\cos\alpha - \pi rm\cos\alpha)}$ . . . (1)

Compressive stress in concrete:

$$c_w = \frac{W}{RJ} \times \frac{1 + \dfrac{J}{2R} - \cos\alpha}{2(\sin\alpha - \alpha\cos\alpha - \pi rm\cos\alpha)} = \frac{W}{RJ}K_w \quad . \quad . \quad (2)$$

Tensile stress in reinforcement:

$$t_w = \frac{m(1 + \cos\alpha)c_w}{1 + \dfrac{J}{2R} - \cos\alpha} \quad . \quad . \quad . \quad . \quad (3)$$

The charts in *Figs.* 2 to 5 are based on these formulæ with $m = 15$, which is the value of the modular ratio commonly used. Separate charts are given for ratios of $\dfrac{J}{R}$ equal to 0·06, 0·10, 0·12, and 0·18.

APPLICATION OF CHARTS.—The procedure for using the charts, assuming that J, R, e, and p (or r) are known or can be determined, is to select the chart from *Figs.* 2 to 5 giving the nearest value of $\dfrac{J}{R}$. Calculate $\dfrac{e}{R}$ and follow along the

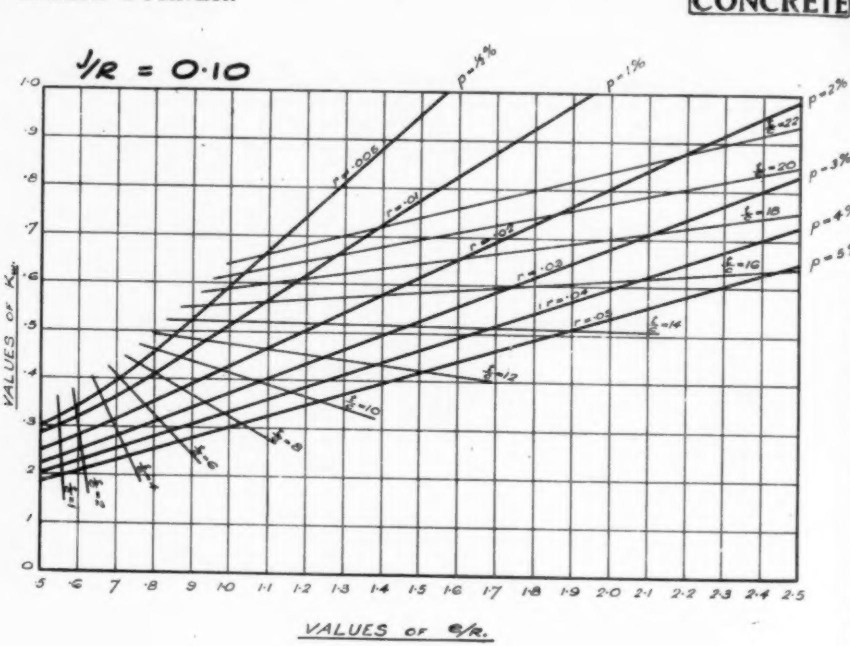

J/R = 0·10
VALUES OF K_w
VALUES OF e/R.
p = 1/2%
p = 1%
p = 2%
p = 3%
p = 4%
p = 5%
r = ·005
r = ·01
r = ·02
r = ·03
r = ·04
r = ·05

Fig. 3.

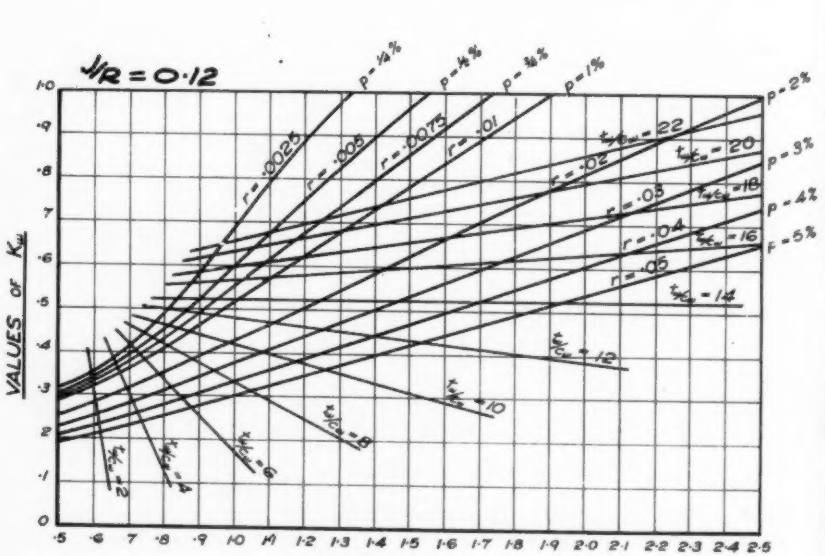

J/R = 0·12
VALUES OF K_w
VALUES OF e/R.
p = 1/4%
p = 1/2%
p = 3/4%
p = 1%
p = 2%
p = 3%
p = 4%
p = 5%
r = ·0025
r = ·005
r = ·0075
r = ·01
r = ·02
r = ·03
r = ·04
r = ·05

Fig. 4.

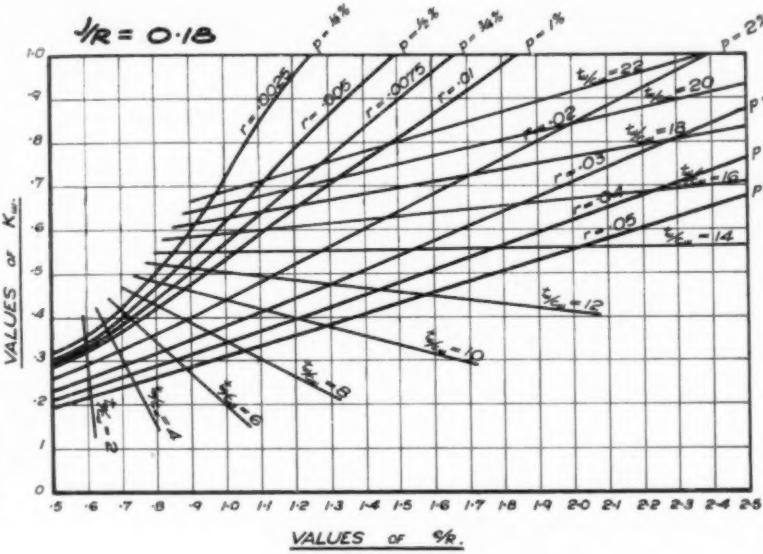


Fig. 5.

abscissæ to the intersection with the appropriate curve of p (or r); read on the chart the corresponding value of K_w and substitute this value in $K_w \dfrac{W}{RJ}$ to give c_w. Also read on the chart the corresponding value of $\dfrac{t_w}{c_w}$ to give t_w.

The values of $\dfrac{J}{R}$ for which the charts are drawn are practical ratios, and after a little inspection and use it is relatively simple to interpolate between them to suit the diameter and thickness of shaft assumed in the first design. If the calculated stresses are too high or too low, a simple adjustment is made to J, or J_s, which are the only variables, and the process is repeated.

Example.

Calculate the stresses at a horizontal plane in the shaft where the weight W is 978,000 lb. and the bending moment M due to the wind is 5,088,000 ft.-lb. The external diameter of the shaft is 17·75 ft. and the thickness is 7 in.

Therefore $e = \dfrac{M}{W} = \dfrac{5{,}088{,}000 \times 12}{978{,}000} = 62{\cdot}5$ in. If the thickness J of the shaft is 7 in., $R = \left(\dfrac{17{\cdot}75 - 0{\cdot}58}{2}\right) = 8$ ft. 7 in. Therefore $\dfrac{e}{R} = \dfrac{62{\cdot}5}{103} = 0{\cdot}606$, and $\dfrac{J}{R} = \dfrac{7}{103} = 0{\cdot}068$. If the vertical reinforcement at this level is 132 $\frac{7}{8}$-in.

bars, the cross-sectional area of the reinforcement is 79·33 sq. in., and

$$r = \frac{J_s}{J} = \frac{79{\cdot}33}{7 \times 2\pi \times 103} = 0{\cdot}017.$$

Using the curves in *Fig.* 2, that is for $\frac{J}{R} = 0{\cdot}06$, stresses slightly lower than the actual stresses will be obtained. For $\frac{e}{R} = 0{\cdot}606$ and $r = 0{\cdot}0175$, read $K_w = 0{\cdot}29$ and $\frac{t_w}{c_w} = 2{\cdot}1$. Therefore

$$c_w = \frac{K_w W}{RJ} = \frac{0{\cdot}29 \times 978{,}000}{103 \times 7} = 395 \text{ lb. per square inch,}$$

and $t_w = 2{\cdot}1 \times 395 = 830$ lb. per square inch.

Using the curves in *Fig.* 3 for which $\frac{J}{R} = 0{\cdot}10$, stresses slightly greater than the actual stresses will be obtained. $K_w = 0{\cdot}30$ and $\frac{t_w}{c_w} = 2{\cdot}2$. Therefore $c_w = 410$ lb. per square inch and $t_w = 900$ lb. per square inch.

Interpolating between the stresses for 0·06 and 0·10 for 0·068, $c_w = 400$ lb. per square inch and $t_w = 850$ lb. per square inch.

Repeated Loading of Beams.

According to the annual report of the Building Research Board ("Building Research 1958." H.M. Stationery Office. Price 5*s*. 6*d*.) tests made at the Building Research Station show that where failure of prestressed concrete beams is due to fatigue of the steel, the ratio of the maximum value of the repeated load just insufficient to cause failure after one million repetitions to the static load causing failure was between 0·65 and 0·85. The lower ratio was obtained with deformed wires and the higher ratio with plain wires that had been subjected in manufacture to a final heat treatment at relatively low temperature. For failure due to fatigue of concrete in compression, the ratio was between 0·77 and 0·82 whether the concrete was made with rapid-hardening Portland cement or with high-alumina cement. Where failure due to fatigue occurred large deflections and extensive cracking developed before collapse became imminent.

Failure under repeated loading of reinforced concrete beams with mild steel reinforcement occurred as a result of yielding of the steel. The ratio of the repeated load just insufficient to cause failure to that causing failure under static conditions was about 0·9. For beams reinforced with cold-worked deformed bars, failure under repeated loading was due to fatigue of the steel; the ratio for these beams was about 0·7.

In the case of prestressed and reinforced beams repeated loading within the range of the working load had little effect on subsequent behaviour, and repeated loading with a maximum load closely approaching that causing failure due to fatigue could be sustained without reducing the strength under static loading.

Tests on prestressed beams under static loading support the suggestion that the permissible bending tensile stress used in design should be related to the initial precompression in the concrete, and indicate that simple relationships can be used for calculating the strength of beams with pre-tensioned wires or post-tensioned and grouted cables or bars provided that the design and construction conform to current practice.

Elevated Road, Chiswick, London.

THE elevated road (*Fig.* 1) which passes over other roads, a roundabout, and railways between the Great West Road and the extension of Cromwell Road has two carriageways each 24 ft. wide separated by a central reservation, and verges about 5 ft. wide along each outer side. Access to the roundabout is by means of four slip-roads.

The part over the roundabout comprises four freely-supported spans 125 ft. long and 59 ft. wide, and a bridge in the approach with a skew span is 70 ft. long and 124 ft. wide. The approaches and the road between the bridges are on embankments retained between plain concrete walls faced with brick. The filling is of gravel containing between 5 per cent. and 15 per cent. by weight of material passed through a No. 200 sieve, and compacted in 9-in. layers by means of 8-tons smooth rollers. After compaction the material had about $7\frac{1}{2}$ per cent. of voids; near the retaining walls and abutments vibrating-plate compactors were used to reduce the voids to about 5 per cent.

The Main Bridge.

The abutments and the two easterly piers of the longer bridge are on reinforced concrete rafts founded on compact gravel overlying London clay. The western pier is on a narrow piled foundation owing to the proximity of a 48-in. diameter sewer 20 ft. below ground. Ten bored piles each 3 ft. 3 in. in diameter and carrying up to 370 tons were constructed to a depth of 80 ft., 60 ft. of which are in London clay. The upper 25 ft. of each pile are reinforced. The abutments are of plain concrete with brick facings and each pier consists of five large columns which are lightly reinforced and a strongly reinforced capping beam (*Fig.* 2).

In each span there are fifteen concrete beams side by side (*Fig.* 2). Each beam was precast in three parts, the outer parts being about 38 ft. long and the central parts 48 ft. long; each part weighs about 33 tons. The strength of the concrete at 28 days was at least 7500 lb. per square inch. The members have mild steel secondary reinforcement and

Fig. 1.

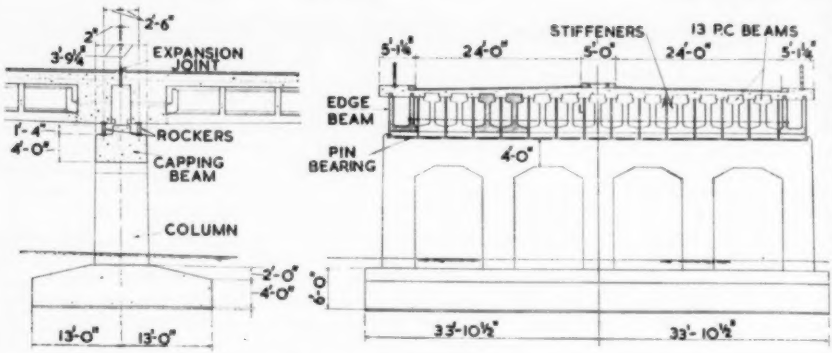


Fig. 2.—Transverse Sections.

were prestressed at a factory by the Freyssinet method. They are of inverted tee-section with a narrow top flange and transverse stiffeners were cast in them. The bottom flange of each beam contains eighteen straight ducts with steel sheaths of 1¾-in. diameter; the ducts terminate against steel plates cast in the ends of the beams and having a corresponding number of holes. The beams at the sides of each span are of U-section (*Fig.* 2); the aggregate is grey Cornish granite, and the exposed vertical faces are lightly bush-hammered.

The carriageway consists generally of a reinforced concrete slab 10 in. thick with an upper layer of 3½ in. of hot-rolled asphalt. Curing of the concrete was by means of a spray of bituminous emulsion. Transverse joints in the slab are at intervals of up to 120 ft., and a strip of steel fabric was laid flat on the slab at the joints before surfacing as a precaution against cracking of the asphalt. The lighting columns are of reinforced concrete and are of octagonal cross section. The road and bridges were designed to comply with the Ministry of Transport Standard Highway Loading and for the Ministry's 180-tons abnormal live load.

Owing to the congested conditions at the site the beams were made in a factory about 60 miles away and transported to the site by road. A 45-tons crane (*Fig.* 3) was used to place them on the bearings on the piers and abutments, and on temporary rollers supported on steel trestles at the transverse joints between the three parts of the beams. These joints are nominally 3 in. wide, and continuity of the ducts across the joints was ensured by short lengths of rubber tube clipped to the ends of the projecting sheaths. The joints were filled with rapid-hardening concrete. When this concrete had attained a crushing strength of 6000 lb. per square inch (about three days after casting) the beams were prestressed by the Freyssinet method. Each duct contains twelve high-tensile wires of 0·276 in. diameter which were tensioned between anchorages bearing on steel plates embedded in the ends of the beam. The initial force applied to each cable was 113,000 lb., and the ducts are filled with colloidal grout applied under pressure.

Fig. 3.—Erecting Precast Beams.

When all the beams in one span had been erected, inflatable rubber tubes were passed into $3\frac{1}{2}$-in. ducts formed in the stiffeners, and concrete was placed in the longitudinal joints and the joints between the stiffeners. A reinforced concrete slab was cast at the level of the top of the beams, and the whole deck was prestressed transversely by cables threaded through the webs and stiffeners of the beams and tensioned by the Gifford-Udall method. The anchorages are in the inner web of the U-shaped beams at the sides of the deck.

"Pinned" or rocker bearings (*Figs.* 4 and 5) are provided on the abutments and piers. The bearing-plates and rockers are of high-tensile corrosion-resisting cast iron and the locating pins are of stainless steel. The expansion joints in the carriageways, verges and reservation, above the rocker bearings, are stainless-steel sliding plates.

The Skew Bridge.

This bridge was constructed in halves longitudinally. The abutments are of plain concrete on reinforced raft founda-

Fig. 4.—"Pinned" Bearings.

Fig. 5.—Rocker Bearings.

tions and have brick facings. The deck comprises forty-three precast beams of inverted tee-section which were prestressed during manufacture by the Freyssinet method. The beams, which were made in the same factory as the beams for the main bridge, are 72 ft. long and each weighs about 24 tons. They were placed side by side on the bearings, and transverse stiffeners were cast between the beams and around metal ducts passing through the webs. The joints between the flanges were filled with concrete and a deck slab was cast at the level of the top of the beams. Each half of the deck is prestressed transversely by the Freyssinet method with cables containing twelve wires of 0·276 in. diameter passing through the transverse ducts.

At the free end each beam rests on a bearing of bonded-rubber and steel 16 in. by 11 in. by 7 in. thick and on a pad of rubber 16 in. by 11 in. by 1 in. thick reinforced with canvas at the fixed end. The greatest force at each bearing is 32 tons due to dead load and 28 tons due to live load. Each bonded-rubber bearing was tested with a force of 30 tons in compression, and also with a shearing force of 8 tons simultaneously with a force of 30 tons in compression. Bonded-rubber pads were used because of the tendency for movements due to expansion

and contraction to occur in longitudinal and transverse directions. The outer beams have grey Cornish granite aggregate and are lightly bush-hammered on the exposed faces.

The consulting architect to the Ministry of Transport was Mr. G. Stewart and the consulting engineer was Mr. Harry Brompton. The main contractors were the Alderton Construction Co., Ltd., the piling was carried out by Braithwaite Foundations & Construction, Ltd., and the precast beams were made by the Cowley Concrete Co., Ltd.

Losses of Prestressing Force.

Mr. B. K. Chatterjee, M.Sc., A.M.I.C.E., of Calcutta, writes as follows.

Sir,—Dr.-Ing. Jerzy Zielinski in his article on " Losses of Prestressing Force " in your number for May 1959, states that the values for the losses of prestressing force when metal ducts are used are about 50 per cent. higher than those given by Mr. Cooley's coefficients which were based on laboratory tests. I have undertaken tests on many prestressed concrete structures of long span and I am of the opinion that the coefficients suggested by Mr. Cooley can be safely recommended for application under site conditions. Reference should be made to an article entitled " Friction Losses in Prestressing under Site Conditions " in " Civil Engineering and Public Works Review ", July, 1956, in which Mr. J. Bobrowsky and myself describe the checking of Mr. Cooley's coefficients under site conditions during the prestressing of the edge-beams of the shell roof of the assembly hall at Kidbrooke Comprehensive High School, London. The cables are in groups and are about 75 ft. and 86 ft. in length. It was realised that in order to avoid errors due to the cables being of parabolic shape the different radii of curvature at various points had to be correctly determined, as described in the article where the results of some tests are given. The Freyssinet system of prestressing was used with twelve wires of 0·2 in. diameter in metal tubes. Moderate vibration during the consolidation of concrete was assumed. Forty cables were tested and the maximum deviation from the theoretical values (using Mr. Cooley's coefficients and our method of dealing with parabolic curves) was only 2·22 per cent. A typical result was a deviation of 2·02 per cent.

Dr.-Ing. Jerzy Zielinski replies as follows.

Mr. Chatterjee's letter gives me the opportunity to point out a specific factor regarding the losses of prestressing force due to friction which I think may be useful in promoting better understanding of the problem.

One of the most important factors influencing the losses of prestressing force in straight cables is the stiffness of the metal sheaths. Since different types of sheath are used in various countries, it was thought that it would be useful to compare the losses obtained with the metal sheaths described by Mr. Cooley with other sheaths. In particular, the metal sheath 0·15 mm. to 0·25 mm. thick produces, as reported, a coefficient of loss (K) per metre length of 1·52 per cent. Nearly the same value of K is obtained in preformed ducts when metal sheaths 0·6 to 0·8 mm. thick are used. Both these values are about 50 per cent. higher than those given by Mr. Cooley (*Table I*, point 2, column 5, in " Estimation of Friction in Prestressed Concrete ", Cement & Concrete Association, 1954), whereas some values of K, for example, for sheaths 0·8 mm. thick, which were probably similar to those used by Mr. Cooley, agree well with those given by him, namely 0·90 per cent. in my tests and 1 per cent. in Mr. Cooley's tests.

Therefore, the results I was able to publish had to justify the opinion that Mr. Cooley's coefficients should be checked when they are used for metal sheaths other than those specified by him. It was never my intention to correct Mr. Cooley's coefficient as he used it, as seems to be implied in the comment by Mr. Chatterjee.

Book Reviews.

"Vibration Engineering." By W. Ker Wilson (London: Charles Griffin & Co., Ltd. 1959. Price 90s.)

THE theory of mechanical oscillation and the reduction of vibration is dealt with in great detail, but in general the emphasis is on the effects of vibration on machines rather than on the supporting structures. In the first part, which occupies about a third of the book, the balancing of machines in order to reduce vibration is considered, but in the second part there is more matter that may be of interest to structural engineers. The isolation of vibrating machines is dealt with first by giving methods of determining the frequency of vibration, followed by descriptions of, and calculations in connection with, anti-vibratory mountings such as suspension links, springs, and rubber pads. In most of the problems discussed it is assumed that the machine and its flexible mountings are supported on a rigid and inelastic foundation, which is an acceptable assumption in general as regards the machine. The author explains that the natural frequency of a block of concrete embedded in the ground may generally be between 1000 and 2000 cycles per minute, but in marshy ground it may be as low as 500 cycles and on rock as high as 4000 cycles. The results can be serious if the frequency of vibration of the machine coincides with that of the ground. Some guide to the avoidance of this condition is given in connection with foundations and machines on upper floors of buildings. The effect of vibrations of the ground being transmitted through the foundations to the machine is also considered.

The principal numerical example is of a small oil-engine driving a generator and founded on a concrete base. The treatment is mainly from the mechanical aspect, but the effect on the ground and means of damping vibrations to avoid causing damage to adjacent structures are considered. A structural engineer might expect to find in this book an example of the design of large elevated substructure for a turbo-generator but, although the basic principles may be contained in the mathematics, this specific problem is not considered. The mathematics is in general so complex that the first chapter is devoted to an explanation of the use of digital computors.

"Die zweiseitig gelagerte Platte." By Hugo Olsen and Fritz Reinitzhuber. (Berlin: Wilhelm Ernst & Sohn. 1959. Price 24 D.M.)

THIS book deals with the design of slabs supported along two edges. Data are provided for the calculation of the bending moments and deflections of freely-supported slabs of any ratio of span to width, based on the differential equation for the loaded slab. It is shown that for a slab having a ratio of span to width of 1 : 4 and carrying a uniformly-distributed load p over the entire area, and assuming Poisson's ratio to be 1 : 6, the bending moment in the unsupported direction is $0{\cdot}02083pl^2$ or $16\frac{2}{3}$ per cent. of the bending moment $0{\cdot}125pl^2$ in the direction of the span l. If distribution reinforcement is provided in accordance with the recommendations of the British code of practice there is sufficient steel to resist the longitudinal moments calculated by the authors' method.

"Bauforschung 1945–1958." Vol. I. Compiled by Deutsches Bauzentrum e.V. (Stammheim-Calw: Lothar Rossipaul. 1959. Price 16 D.M.)

THIS is a list of references to German literature on building research during the period 1945 to 1948. Publications concerned with concrete and cement are listed under Cement, Mortar and concrete, Aggregates, Admixtures, Porous concrete, Lightweight concrete, Testing, Precast concrete, Construction, and Prestressed concrete. Only the names of the authors and publishers and the title of the publication are given.

"Zur Theorie und Berechnung von Schalentragwerken in Form gleichseitiger hyperbolischer Paraboloide." By Werner Bongard. (Berlin: Wilhelm Ernst & Sohn. Price 7.60 D.M.)

THE theory of thin saddle-shaped structures of hyperbolic paraboloidal form and the calculation of stresses in them are described in this thesis. An approximate theory of bending is developed, and equations which require the use of an electronic computer for their solution are derived for use in calculating deformations, bending moments, and shearing forces.

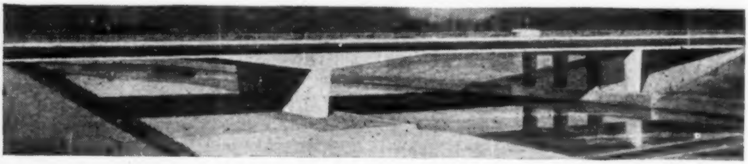


FIG. 1.

Prestressed Bridge over the River Medway.

PRECAST SUSPENDED GIRDERS WEIGHING 165 TONS.

A BRIDGE (*Fig.* 1) 325 ft. long and 87 ft. wide to carry the Maidstone by-pass road over the River Medway is being constructed of prestressed and reinforced concrete. The deck comprises a balanced cantilever, two suspended spans, and an end span with a short cantilever.

The main span is 145 ft. long. There will be two carriageways each 24 ft. wide, a central reservation 15 ft. wide, and two pavements each 8 ft. wide. The lack of symmetry of the bridge is caused by the difference of 40 ft. between the levels of the banks of the river and by the requirement that a tow-path at least 15 ft. wide be provided on one bank. It was also required that provision be made for a possible future widening of the river and that no temporary supports be placed in the river during construction. The height of the bridge is controlled by the need to provide clearance over an adjacent railway which is considerably higher than the river.

The deck (*Fig.* 2) comprises slabs 10 in. thick supported on four hollow girders of prestressed concrete with internal transverse walls at intervals of about 10 ft. and which are stiffened by transverse reinforced concrete beams cast in place at intervals of about 20 ft. along the length of the bridge. The walls of the girder vary in thickness between 1 ft. 2 in. and 1 ft. 6 in., and the top and bottom flanges generally vary in thickness between 10 in. and 1 ft. but are thicker near the supports.

At the east end of the bridge an open type of abutment (*Fig.* 3) is provided on which are supported the ends of four prestressed girders about 81 ft. long. To enable the prestressing cables to be tensioned, the girders are to be cast over their final position and later lowered on to their

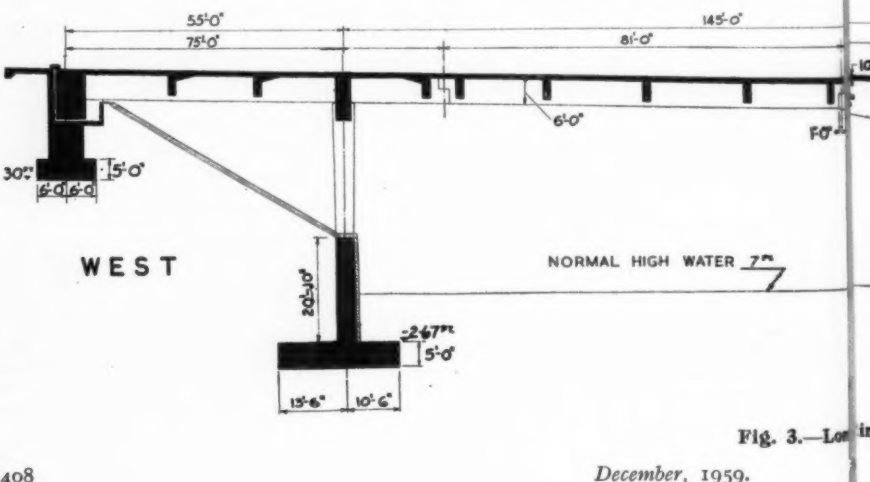


Fig. 3.—Lo

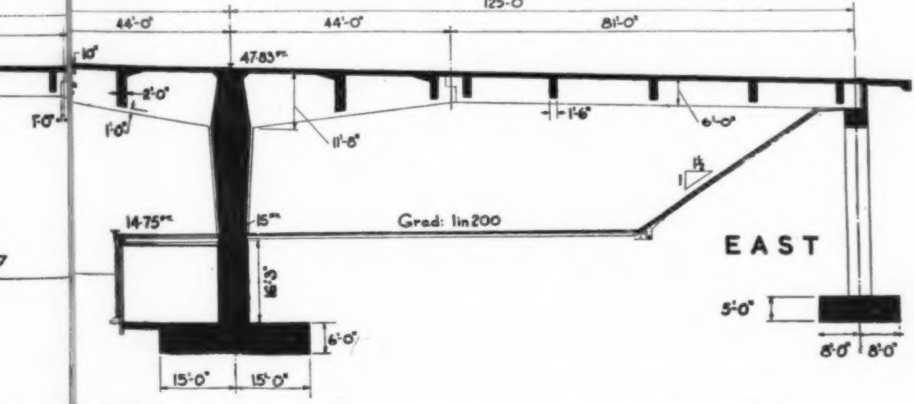
supports by means of jacks. On the east bank a balanced cantilever is supported on a reinforced concrete pier founded on a reinforced concrete strip footing resting on Kentish ragstone below the bed of the river. The girders for the central suspended span will be precast near the bridge and rolled into position over a temporary structure, probably a Bailey bridge. These girders will weigh 165 tons each, and are said to be the biggest to be made and erected in this way. The girders of the other span rest on bearings at the top of the western embankment and are supported on, and cantilever 20 ft. beyond, a pier at the western bank of the river.

The piers are to be constructed within steel sheet-pile cofferdams. Part of the piling on the eastern bank is to be left in place and faced with masonry and will retain the tow-path. The piling is to be tied to the footing and to the pier by horizontal 2-in. bars at intervals of about 5 ft. along the top and base of the wall, since the piles cannot be driven into the rock. The eastern pier is of reinforced concrete and is about 50 ft. high from the top of the footing to the top of the girders.

The girders forming the balanced cantilever extend about 44 ft. on each side of the pier and are to be prestressed in three stages with sixty-two cables each comprising twelve wires of 0·276 in. diameter. The girders will be 8 ft. wide, about 11 ft. 8 in. deep at the junction with the piers, and 5 ft. 10 in. deep at the ends. In the first stage of construction the girders are to be cast complete and the pier and transverse beams are to be cast to within 1 ft. 7 in. of the top. Thirty-six of the prestressing cables will then be tensioned, and will extend throughout the entire length of the cantilevered girders. The suspended girders will then be placed in position and the slabs cast on them. The slabs on the cantilevers will then be cast. In this way the bending stresses produced by the suspended spans in the slabs of the cantilevered spans will be due to the roadway and imposed loads only.

In the second stage the slabs are to be completed for a distance of 23 ft. on each side of the pier, that is as far as the first transverse stiffeners. Nine cables in each beam and nine alternate cables in each slab will be anchored at these stiffeners

Fig. 2.—Typical Cross Section.



3.—Lo inal Section.

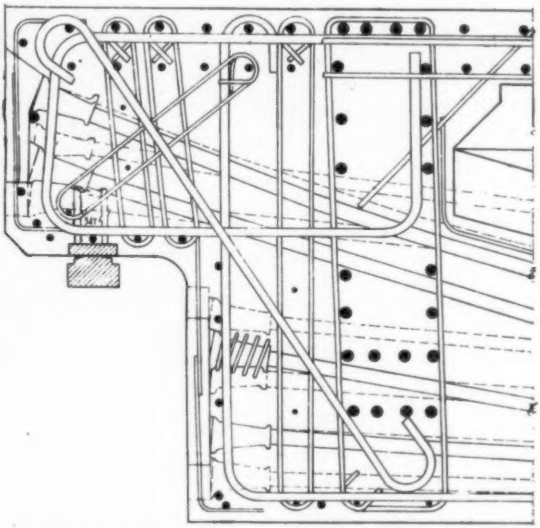


Fig. 4.—Typical Reinforcement at End of Main Girders.

and will then be tensioned. The concreting will then be completed and the remaining cables tensioned at the transverse beams about 40 ft. from the pier. The cables anchored at the transverse beams will be curved downwardly for the last 10 ft. to anchors in the sides of the beams below the soffit of the slab. The cables anchored in the ends of the girders will be curved downwardly to provide resistance to shearing. Manholes will be provided for access to the anchorages inside the girders. Mild steel reinforcement will be provided to resist the stresses due to shrinking and changes of temperature, and to resist the bursting forces around the anchorages (*Fig.* 4).

The girders supported between the balanced cantilevers and cantilevered part of the end girders will be prestressed by forty-six cables each containing twelve wires of 0·276 in. diameter. Most of the cables are curved upwardly near the ends of the beams to resist shearing forces. These beams will be provided with mild steel rocker bearings at each end. Roller bearings will be provided at the abutments and at the top of the western pier. The embankments are to be faced with precast slabs with exposed-aggregate surfaces, and the visible sides of the main beams and piers are to be point-tooled.

It is planned to complete the bridge during a period of fifteen months and the tendered price is about £220,000. The bridge has been designed for the Ministry of Transport and Civil Aviation by Messrs. Scott & Wilson, Kirkpatrick & Partners (who are also supervising the construction) in collaboration with Messrs. Adams, Holden & Pearson acting as consulting architects. The contractors are Sir Robert McAlpine & Sons, Ltd., and the prestressing equipment is supplied by P.S.C. Equipment, Ltd.

Congress on Prestressed Concrete.

The fourth congress of the Fédération Internationale de la Précontrainte will be held in Rome and Naples in June or July, 1962. The subjects to be dealt with include the results of research since 1958, problems at sites, accidents, fire resistance, buckling, stress corrosion, the economics of prestressed concrete structures, progress in precast manufacture and standardisation since 1958, and outstanding prestressed concrete structures constructed since 1958.

UNESCO Headquarters in Paris.

THERE are several examples of exposed concrete surfaces as architectural features in the congress hall and office building at the new headquarters of UNESCO in Paris. To obtain the finish required the use of plywood and metal shuttering was prohibited. Many of the surfaces show intentionally the marks of the joints between the narrow boards used for the shuttering.

The hall (*Fig.* 1) is trapezoidal in plan and is 154 ft. long and tapers from 197 ft. to 111 ft. 6 in. in width. The roof and the end walls are constructed of twelve double-bay rectangular frames the main members of which are basically of vee-section, and are carried on a central support comprising a transverse beam supported by a row of columns. The concrete in the spaces between the members was cast after most of the shrinking of the main members had taken place. A thin curved slab interpenetrates the roof members over the auditorium. The surfaces of the end wall (*Fig.* 2) and roof of the auditorium have a pattern formed by the joints between shutter-boards $2\frac{1}{2}$ in. wide. The wall was cast in 4-ft. 6-in. lifts, and the top of each lift is emphasised by a horizontal rebate. The

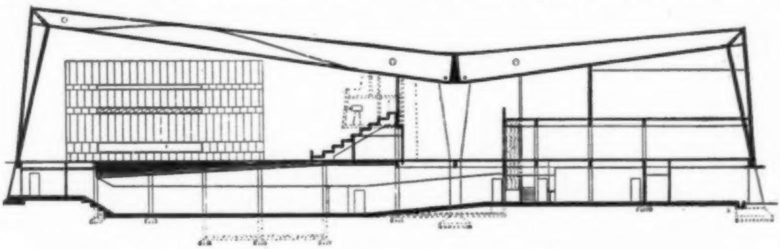


Fig. 1.—Section through Main Hall.

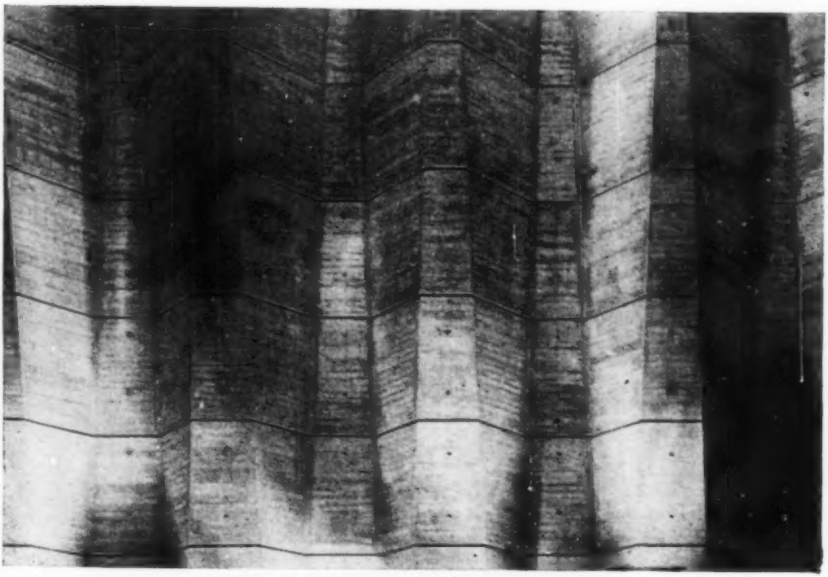


Fig. 2.—Wall behind the Rostrum.

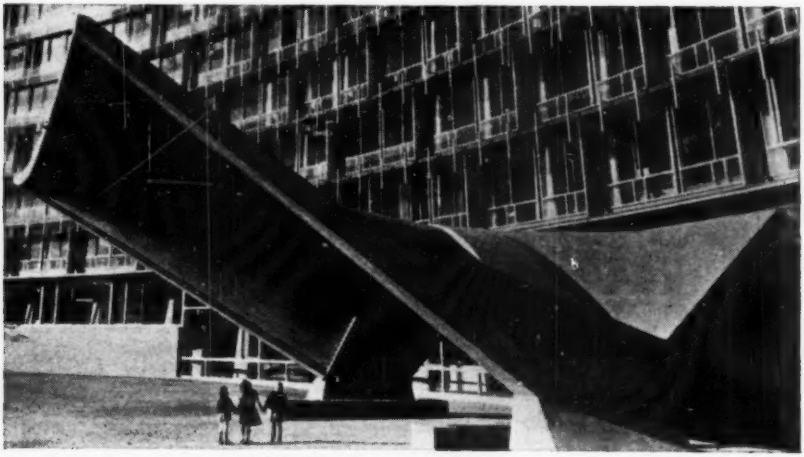


Fig. 3.—Canopy at Entrance.

columns in the foyer, which are of circular cross section at the base and rectangular at the top, have been lightly rubbed and the marks left by the narrow shutter boards are faintly visible. The walls of the entrance to the auditorium are bush-hammered.

The office building comprises seven upper stories supported on columns in the ground-floor story, the shape of the columns varying from a narrow pentagon at the top to an ellipse at the bottom as shown in *Fig.* 4. The narrow shutter-boards for the columns were so placed that the joints formed a pattern of horizontal lines on some faces and vertical lines on others. The canopy over one of the entrances (*Fig.* 3) comprises two thin doubly-curved slabs, which are cantilevered, one on each side, from a central arch. The surfaces of the curved slabs exhibit the marks of the joints of the narrow shutter-boards which are parallel to the longitudinal axis of entrance.

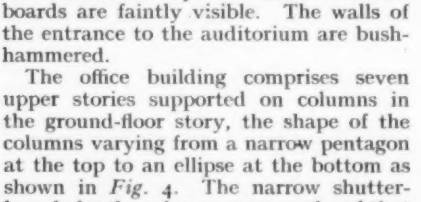


Fig. 4.—Columns in Ground Story.

The buildings were designed by M. Breuer, B. Zehrfuss, and P. L. Nervi. The contractors were Fourré and Rhodes-Dumez. The foregoing and the accompanying illustrations are from the "Concrete Quarterly" for January-March, 1959.

Measuring Temperatures in Concrete.

An investigation of the use of thermistors for measuring temperatures in concrete has been made by Mr. Knud E. C. Nielsen at the Swedish Cement and Concrete Research Institute, and the results are given in a bulletin entitled "Temperature Measurements with Thermistors in Concrete" (price Kr. 8). Chapters deal with the properties of thermistors and methods of measurement, and examples of their use are given. It was found that the temperatures recorded were accurate to about 0·1 deg. C., and that the thermistors were cheap, simple to calibrate and to use, and were not damaged when they were used on construction sites. The tests included measurements of the temperature in the interior of (a) concrete subjected to alternate freezing and thawing, (b) concrete subjected to loading (in order to ascertain the internal stresses), (c) concrete placed under water (in order to ascertain the time of setting so as to determine a suitable rate of placing further concrete).

Bridges on the London-Birmingham Motorway.

THE construction of much of the London–Birmingham motorway, which was described in this journal for October last, is now complete. Fifty-five miles of dual carriageway, including more than 120 bridges from near Luton to near Rugby, are now open to traffic and were constructed in nineteen months. Thirteen of the bridges to the south-west of Northampton were constructed by sub-contractors and comprised two wall-and-slab viaducts, an arch bridge over a canal, a bridge over the railway, and nine two-span bridges over the motorway, and these are briefly described in the following.

At the crossing of the River Nene there are two viaducts with reinforced concrete decks carried on ten piers, each 120 ft. long. The span at the river crossing is 56 ft. between the piers; the remaining ten spans are each of 28 ft. The deck over the side spans is 1 ft. 9 in. thick, with the exception of the slabs on either side of the long span which are 3 ft. thick and are extended to cantilever 14 ft. over the river from each side. A slab 28 ft. long over the river is carried on the two cantilevered slabs.

Plain concrete foundations extend to 15 ft. below the level of the ground and support the reinforced concrete piers, which vary in thickness from 2 ft. 10 in. at the bottom to 1 ft. at a height of 16 ft. Openings with semi-circular heads (*Fig.* 1) are formed in the piers to reduce their weight. The viaduct over the relief channel is similar in detail but has nine equal spans. Although the river and the relief channel are narrow, these viaducts have been provided to span the flood plains on either side, and are 376 ft. and 357 ft. long respectively. Severe flooding was encountered during construction.

The bridge over the Grand Union canal at Milton Ham is of plain concrete and has a span of 40 ft. It is supported on reinforced concrete piles up to 60 ft. long. The soffit of the arch is 32 ft. above the towpath. The motorway is on an embankment about 50 ft. above the level of the natural ground. The wing walls are 204 ft. long. The fascias of the arch are parabolic on plan. Construction of the arch was carried out by means of a travelling shutter.

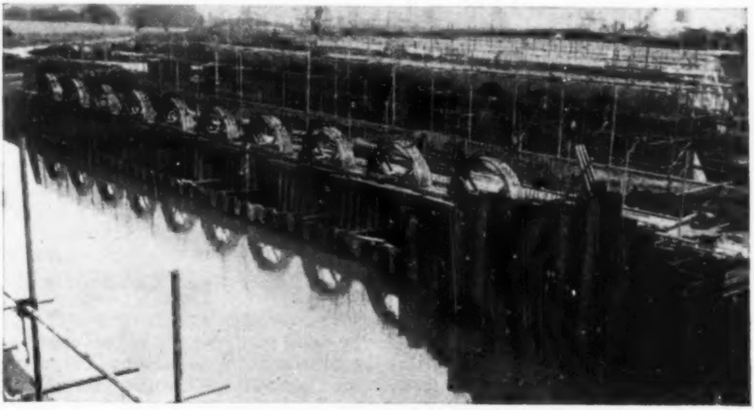


Fig. 1.—Construction of Piers of Nene Viaduct.

Two of the two-span over-bridges of standard design to carry existing roads over the motorway form part of an elevated roundabout, where slip-roads lead on to and out of the motorway. To allow for the different angles at which the motorway is crossed the lengths of the abutments and lengths of the spans are different, although all the bridges on the

Fig. 2.—Bridge over a Railway.

major and minor roads have a similar appearance. The widths of the decks are from 32 ft. to 42 ft. 6 in., the square span between the abutments is 123 ft. in all cases, and the lengths of the abutments are up to 95 ft. The superstructures of the bridges are entirely of reinforced concrete. The foundations are of plain concrete, but in some cases the structures are founded on precast reinforced concrete piles. The cylindrical central piers are 4 ft. 6 in. in diameter and support beams which carry the deck, which is generally 2 ft. thick.

The railway bridge was constructed in such a way that interference with the use of the railway was as little as possible. Plain-concrete abutments and reinforced concrete beams carried on cylindrical columns support precast reinforced concrete inclined beams as shown in *Fig.* 2. The beams form the soffit of a sloping reinforced concrete slab on each side of the bridge and which is cantilevered over the tracks. Precast reinforced concrete deck beams were then placed to span between the two cantilevers and form the soffit of a reinforced concrete slab cast in place by means of which the structure is made monolithic. The tarmacadam surface of the road was laid directly on this slab. (This type of bridge is described in this journal for March 1958 and October 1959.)

The thirteen bridges included in this sub-contract were commenced in May 1958, and completed in January 1959, before the road-making machines reached each site. Each over-bridge was constructed in about twenty weeks, and the other structures in twenty to thirty weeks each. Shuttering faced with plywood was used throughout.

The consulting engineers to the Ministry of Transport and Civil Aviation are Sir Owen Williams & Partners. The main contractors were Messrs. John Laing & Son, Ltd., and the sub-contractors for the thirteen bridges described in the foregoing were Messrs. Holloway Brothers (London), Ltd.

Rapid Road Construction.

THE Watford branch of the St. Albans by-pass road, which is the continuation southwards of the Rugby–Luton part of the London to Birmingham motorway, has been completed several weeks before the scheduled date. The branch comprises dual concrete carriageways 26 ft. wide, the construction of which was described in this journal for October, 1959. It is claimed that a record for concrete road construction in Great Britain was established by laying 1320 ft. of 26-ft. carriageway in eleven hours. The total cost of the Watford branch is about £1,250,000, and the value of the equipment installed to hasten construction was nearly £1,000,000. The contractors for the Watford branch are a consortium comprising Messrs. Holland & Hannen and Cubitts (Great Britain), Ltd., Messrs. Fitzpatrick & Sons (Contractors), Ltd., and Messrs. Lehane, Mackenzie & Shand, Ltd.

A New Type of Cast-in-Place Pile.

FIVE hundred piles of a new type (known as Delta) are used to support oil-storage tanks and a boiler-house at the depot of Shell Mex & B.P., Ltd., at Wandsworth, London. The piles are 30 ft. long and are founded on ballast underlying poor clay and made-up ground.

The piles are of a Swiss type introduced recently into this country, and may be reinforced or not. The stages in the construction of an unreinforced pile are indicated in *Fig.* 1 and are: (1) Pitch the driving tube on a reinforced concrete shoe with a small quantity of concrete between a steel mandrel and the shoe. (2) Drive the tube to the required level or set by means of a diesel-driven hammer. (3) Insert sufficient concrete in the tube to form a bulb. (4) Using the hammer and mandrel, ram the concrete while withdrawing the tube slightly, thus forming a bulb of concrete. (5) Fill, or partly fill, the tube with concrete, and, with the hammer and mandrel resting on the concrete, withdraw the tube. (6) Insert reinforcement to connect with the superstructure if required. Operation No. 5 may be carried out in one step for the entire pile or in several steps for longer piles. If the pile is to be reinforced, the method is basically as described, but four bars are attached to the mandrel between operations Nos. 3 and 4. The bars are attached to the mandrel with wire which breaks when starting to ram to form the bulb.

To take advantage of good intermediate bearing strata, extra bulbs can be made on the pile by interrupting operation No. 5 at the required depth and repeating operations Nos. 3 and 4. If the bearing stratum is poor, larger bulbs can be produced by introducing extra concrete in operation No. 3 and withdrawing the tube correspondingly farther in operation No. 4.

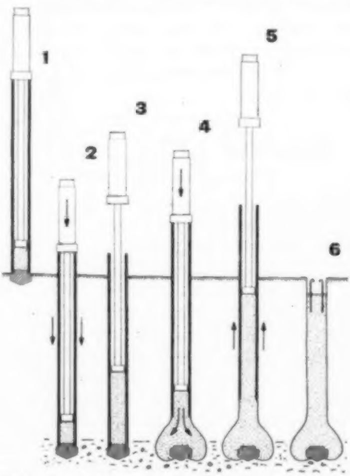


Fig. 1.—Stages of Forming a Pile.

Fig. 2.

For longer piles the tube and mandrel can be lengthened.

Fig. 2 shows a pile being formed. The tube, which is partly filled with concrete, is being withdrawn after the formation of the bulb and while the hammer and mandrel are resting on the concrete. The diameter of the shoe is 1 ft. 7 in., and the nominal diameter of the shaft is 1 ft. 4½ in., which is the external diameter of the tube; the size of the shaft is likely to be larger due to the fact that the concrete is forced into the ground surrounding the tube. The tube is 39 ft. or 42 ft. long, but can be extended to 65 ft. or 75 ft. The diameter of the bulb is not less than 3 ft.

The licensees for the system are H.D. Foundations, Ltd.

Hydrographical Folly from B.S.I.

"Tests for Water for Making Concrete." British Standard No. 3148:1959. (London: British Standards Institution. Price 3s.)

A SIMPLE method of ascertaining the suitability of water for making concrete would often be useful, but it is not given in this document. The committee of representatives of two research organisations, six professional institutions, and three trade associations which studied the problem say that it has failed to find a satisfactory method, and instead is content to recommend that specimens be made of cement paste to determine the initial setting-time and of concrete to determine the crushing strength, and that the results be compared with similar specimens made with distilled water. This is no doubt what their grandfathers did. In an appendix it is stated that the committee "considers" that the water will be suitable if the initial setting-time does not differ by more than thirty minutes from that of the specimen made with distilled water, and if the crushing strength at 28 days is not less than 80 per cent. of that of the specimens made with distilled water —but it is only "believed" that the strength of such concrete will not subsequently decrease by reason of the quality of the water. So, after making the specimens and waiting for twenty-eight days to know the results of the crushing tests, the effect of the doubtful water is still not known.

The whole of the useful information in this document is given in the foregoing, with the exception of references to five other British Standards that must be bought and consulted for guidance in making and testing the specimens. If the useless, and sometimes laughable, matter were omitted the standard could have been printed on one side of a sheet of paper and sold for a few pence. Seldom can even the British Standards Institution have offered so little information at so high a price. The fecundity of the Institution is indicated by the fact that this Standard is numbered 3148. This one at least is unnecessary, for it is quite wrong to call it a standard or a specification when the comments in the appendix indicate that the committee has no knowledge of the validity of the recommendations it makes.

A French standard issued in the year 1941 gives recommendations for the acceptable limits of matter in suspension (2 grammes per litre) and of dissolved salts (15 grammes per litre), and a still earlier U.S.A. recommendation gave permissible values for the acidity and alkalinity of water for making concrete. This is the kind of guidance that is wanted, and which this so-called standard specification fails to give.

Measuring the Air Content of Concrete Mixtures.

AN apparatus for measuring the quantity of air in concrete mixtures, and the specific gravity and the amount of free moisture in aggregate, is now available. The container has a capacity of ¼ cu. ft. of the material to be tested, and so provides a means of measuring its density. A nomograph supplied with the apparatus enables the specific gravity and the percentage of free moisture of the aggregate to be determined. The principle used is Boyle's law. The apparatus weighs about 12 lb. and is obtainable from Soiltest, Inc., Chicago, U.S.A.

Accelerated Tests for Cement.

MR. T. N. W. AKROYD writes as follows.

The Editorial Note in your October, 1959, number on an accelerated test for cement was provocative and thought-stimulating, as are many of your editorials.

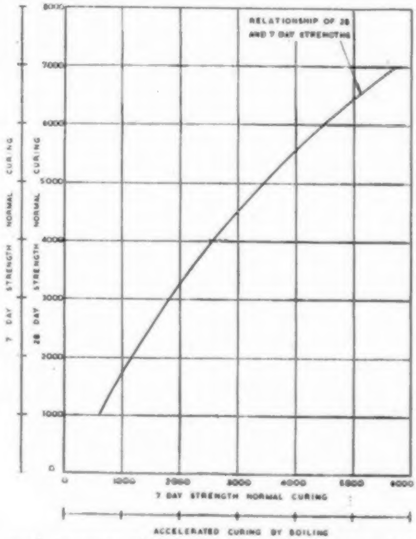
The test you describe appears to be limited to cement, whereas there exists an accelerated-cured compression test for concrete by which the strength of a wider range of concrete can be predicted. In addition, this test has the advantage that it relates to British Portland cements.

In November, 1955, Professor J. W. H. King described the oven-method of accelerated curing, and in February, 1956, Mr. R. G. Smith-Gander and the writer described their method whereby concrete cubes are boiled in water. I advocate the accelerated test by boiling concrete cubes in water because it is simple and easy to perform.

The boiling test is made as follows.

Half an hour after mixing and casting the concrete cubes in cast-iron moulds a steel plate is placed over the top of the cubes, which are then placed in a thermostatically-controlled curing tank where they remain in their moulds for twenty-four hours. At the end of this time they are removed from the tank and, in their moulds, are plunged into boiling water and boiled for 3½ hours. They are then removed from the boiling water, the moulds removed, and the density of each cube measured, and one hour after they are removed from the water the cubes are tested in compression.

When the original accelerated-curing test was devised it was necessary to prepare a separate graph for each type of cement and aggregate. A series of accelerated-curing tests had to be made to relate the accelerated-cured strength to the strength at seven or twenty-eight days. With the procedure described, however, in the case of most Portland cements the accelerated strength bears the same relationship to the seven-days' strength as does the seven-days' strength to the strength at twenty-eight days, so that knowledge of the strengths at seven and twenty-eight days only is required. This may be obtained from actual results or from the graph, which has been prepared from a graph relating the water-cement ratio to the strengths at seven



and twenty-eight days in the report by Mr. H. C. Erntroy, "Variation of Works Tests Cubes", published by the Cement & Concrete Association.

Effect of Calcium Chloride on Steel Reinforcement.

IN concrete made with ordinary Portland cement and containing 2 per cent. of flaked calcium chloride (about 1·4 per cent. anhydrous calcium chloride), cured at normal temperatures with subsequent storage up to one year outdoors, only some discoloration and very slight local corrosion was found. The amount of corrosion increases as the calcium-chloride content is raised, or when specimens are cured and stored at higher temperatures. —"Building Research 1958." H.M. Stationery Office. Price 5*s*. 6*d*.

A Code for Prestressed Concrete.

B.S. Code of Practice No. 115 (1959), "The Structural Use of Prestressed Concrete in Buildings" (price 8*s*. 6*d*.), was published last month by the British Standards Institution. The code includes clauses on definitions and symbols, materials, stresses, losses, design of beams and compression members, statically-indeterminate structures, tensioning systems, workmanship, and testing.

Alteration of Tenders.

THE Joint Consultative Committee of Architects, Quantity Surveyors, and Builders has issued a Code of Practice for Selective Tendering, with sections on The list of tenderers, Invitations to tender, Tender documents, Form of tender, Time for tendering, Opening tenders and notifying results, and Examination and adjustment of the priced bills.

In an introduction it is stated that in no circumstances should a tendered price be altered, and the following clause is included for guidance in cases where errors in pricing need correction before a tender is accepted.

"The examination of the priced bills of quantities should be made by the quantity surveyor who should treat the document as strictly confidential: on no account should any details of the tenderer's pricing be disclosed to any person except the architect unless with the expressed permission of the tenderer.

"The first object of the examination is the detection of errors of an extent that might cause the tenderer, when he is aware of them, to withdraw his tender. If the quantity surveyor finds such errors he should report them to the architect, who should indicate to the tenderer the value of the errors and give him the opportunity of confirming or withdrawing his offer. If the tenderer withdraws, the priced bills of the second lowest should be examined. When a tender is found to be free of serious error, or the tenderer is prepared to stand by his tender in spite of an error, then the architect should inform the tenderer of its acceptance: before doing so, the architect would be well advised to obtain authority from the building owner.

"The method of adjusting any such errors should be to add an endorsement to the priced bills indicating that all rates or prices (excluding preliminary items, prime cost and provisional sums) inserted therein by the tenderer are to be considered as reduced or increased in the same proportion as the corrected total of priced items exceeds or falls short of the original total of such items. This endorsement should be signed by both parties to the contract."

Copies of the Code are obtainable (price 2s.) from the Committee at 82 New Cavendish Street, London, W.1.

Chimneys 500 ft. High at a Generating Station.

THE tallest reinforced concrete chimneys constructed up to the present in Great Britain are two chimneys 500 ft. high (*Fig.* 1) at the generating station of the Central Electricity Generating Board at Northfleet, Kent. The internal diameters are 38 ft. at the bottom and 23 ft. at the top. The thickness of the wall of the shaft is 1 ft. 7 in. at the bottom and 6 in. for a distance of 175 ft. from the top. The chimneys are on raft foundations. Cement of khaki colour is used in the concrete. The external faces are coated with acid-resistant paint for a short distance from the top. The consulting engineers are Messrs. L. G. Mouchel & Partners and the contractors Messrs. Tileman & Co., Ltd.

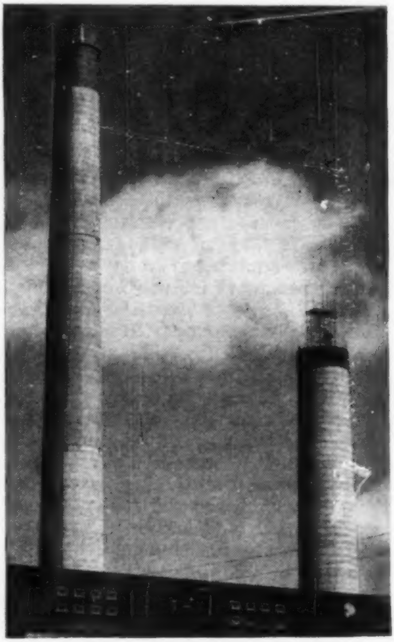


Fig. 1.

A Prestressed Precast Bunker.

A COAL bunker (*Fig.* 1) 90 ft. long, 16 ft. wide, and 15 ft. deep, having a capacity of 500 tons and built recently at Girimint Colliery in West Bengal, is constructed of precast prestressed concrete units assembled and tied together with post-tensioned cables. The bunker spans a rail track and has a clear height of 16 ft. 7 in. above the rails. The two rows of eleven columns are 16 ft. apart transversely. The columns are at 9 ft. centres longitudinally, and were precast and set in pockets 9 in. deep in the foundation. Each column is 18 in. by 9 in. in cross section and 39 ft. 6 in. long, and is in three parts placed end to end through which pass four cables each comprising three wires.

Each transverse beam is in four parts each 4 ft. deep and 9 in. wide through which pass four cables each comprising twelve wires. The beams are carried on corbels on the columns and a cable of eight wires passes through each beam and its supporting columns and ties the beam and two columns together to form a frame. The longitudinal beams are 1 ft. 6 in. wide and comprise successive lengths of 9 ft. each carried on corbels on the columns. A cable of eight wires, which passes through the beams and columns, extends the entire length of the bunker. The wires in all the cables are 0·276 in. diameter and the tensile stress after an assumed reduction of 15 per cent. is 65 tons per square inch. Colloidal cement grout was injected into the ducts for all the cables.

Precast floor slabs 2 ft. 6 in. wide and 7 in. thick are supported on the transverse beams. Openings in the slabs are provided for the outlets, the steel gates of which are 2 ft. square.

The walls are of precast prestressed slabs 1 ft. 9½ in. wide set in grooves in the faces of the columns. The slabs in the longitudinal walls are 3 in. thick in the lower part and 2¼ in. thick in the upper part. The slabs in the end walls are 5½ in. and 4 in. thick. The floor and wall slabs are prestressed by pre-tensioned wires 0·159 in. in diameter which were obtained by unravelling steel-wire ropes available from collieries. The stress in these wires is 63 tons per square inch after

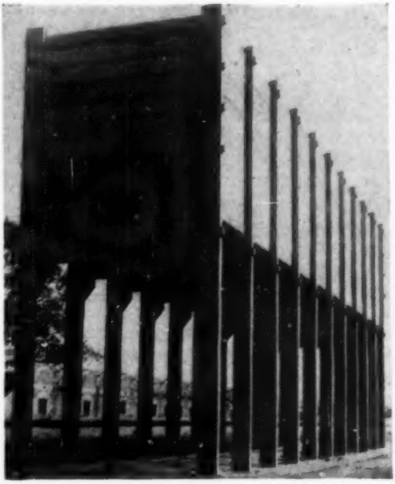


Fig. 1.

allowing for a reduction of 20 per cent. Half-round grooves along the top and bottom edges of the wall slabs accommodate a 0·276-in. post-tensioned wire in alternate horizontal joints and passing through the columns. Each pair of columns is tied by a transverse ¾-in. mild steel bar at each of the quarter-points of the depth of the bunker. A transverse beam, prestressed by a cable containing eight wires, extends across the bunker between the tops of each pair of columns and carries a conveyor.

The crushing strength of the concrete was 5000 lb. per square inch at twenty-eight days. Under ordinary working conditions the calculated maximum compressive stress in the concrete is 1650 lb. per square inch and there is no tensile stress, but should the coal be stored to a height of 5 ft. above the normal level the tensile stress in the concrete of the wall slabs would be 250 lb. per square inch.

The consulting engineer is Mr. S. P. Banerjee of Calcutta. The contractors were the Indian Patent Stone Co., Ltd., who also made the precast members. Gifford-Udall anchors were used for the cables.

Glass-fibre Shuttering.

Moulds made of glass-fibre were used for the shuttering of columns with splayed heads (*Fig.* 1) cast in place at premises for the Gravesend Co-operative Wholesale Society, Ltd. A mould for an interior column comprises two identical parts (one of which is shown in *Fig.* 2) which are fixed together by ⅜-in. bolts at 9 in. centres through the abutting flanges of each half. For a wall column one half only is used, with timber to form the flat surface.

The halves of the mould are formed on a plaster model of half a column. The model is laid on a floor with the curved surface uppermost and is treated with three coats of shellac. The shellac is polished with wax and painted with polyvinyl-alcohol in order to prevent the mould adhering to the model. A layer of polyester-resin is then applied with a brush, and a sheet of glass-fibre weighing 1½ oz. per square foot is pressed into the resin with a roller. Alternate layers of resin and glass-fibre are applied until the mould is of a thickness such that it weighs about 6 oz. per square foot. Steel angles are embedded in the edges of the mould to strengthen the flanges, and a piece of steel strip is partly embedded in it to strengthen it longitudinally. Further pieces of steel strip are partly embedded in the other direction, and also in the part to form the flared top. The inner face is polished.

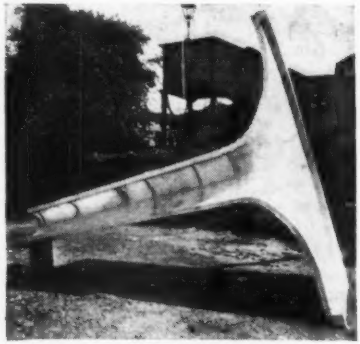


Fig. 2.—Half a Mould.

The moulds are light in weight, non-corrosive, and translucent, and do not require to be cleaned after use. The surface of concrete cast in the moulds (*Fig.* 3) is smooth and is free from blemish if the concrete is consolidated by vibrators. The moulds are said to offer great resistance to abrasion, do not become distorted, can be used many times, and can be quickly assembled and dismantled.

The architect for the building is Mr. W. J. Reed, F.R.I.B.A., and the contractors were the Co-operative Wholesale Society, Ltd. The moulds were made by D.A. Models, Ltd.

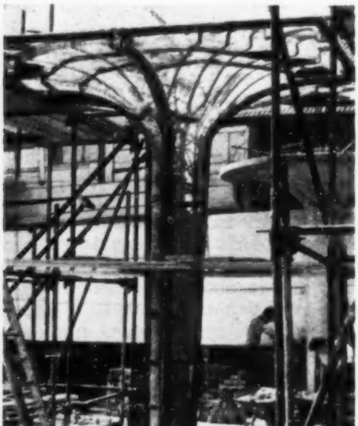


Fig. 1.—Mould ready for Concreting.

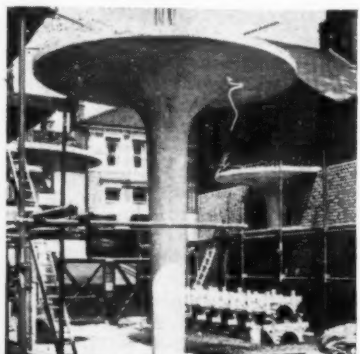


Fig. 3.—Completed 12-ft. 6-in. Columns.

A Plant for Cooling Aggregate.

In order to ensure that the temperature of the concrete does not exceed 55 deg. F. when it is placed, the aggregate used in the construction of a dam across the river Savannah in Georgia, U.S.A., is cooled in tanks of 11 ft. diameter in which during operation the pressure is reduced to less than atmospheric. The plant (*Fig.* 1) comprises three tanks 86 ft. high for the coarse aggregate and two tanks 62 ft. high for the sand. The aggregate is delivered into the top of the tanks by a conveyor. Baffles in the larger tanks distribute the aggregate throughout the vessels and reduce the abrasion against the walls. Baffles in each of the smaller tanks distribute the material in the form of cylindrical curtains of sand with air spaces between the wall of the tank and the sand.

When the tanks are charged, they are sealed and refrigeration commences. Two boilers fired with natural gas generate 34,000 lb. of steam per hour at an operating pressure of about 125 lb. per square inch. The steam is passed through a Venturi tube and the resulting reduction of pressure to about $\frac{1}{2}$ lb. per square inch is transmitted through pipes to the interior of the tanks. Because of the reduced pressure the aggregate is cooled by about 40 deg. F. in 45 minutes.

After it has cooled, the aggregate is taken by conveyor to the batching plant and passed through screens before entering the mixer. Water for the concrete is cooled by a similar process. Cold water is circulated around the storage bins of the batching plant, although the aggregates in the bins gained only 4 deg. F. in temperature in 24 hours in warm weather when no water was circulated.

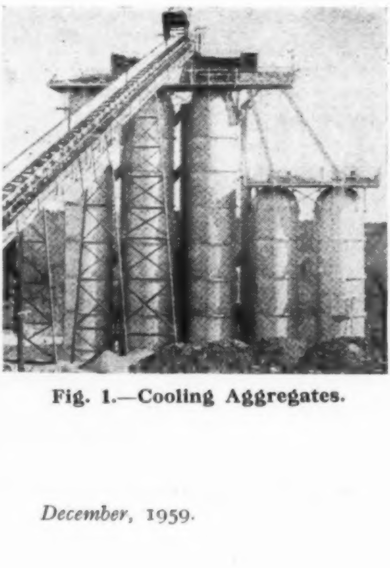


Fig. 1.—Cooling Aggregates.

The plant is capable of cooling enough aggregate and water to produce 250 cu. yd. of concrete per hour. The tanks contain about 600 cu. yd. of coarse aggregate and 60 cu. yd. of sand and an average cooling cycle lasts about 45 minutes. These notes are abstracted from "Engineering News-Record" for April, 1959.

Lectures on Building

The following lectures have been arranged by the Ministry of Works. Admission is free.

Work Study in the Building Industry, by R. Geary. School of Building, Ferndale Road, London, S.W.4. December 15. 7.15 p.m.

Weathering and Deterioration of Concrete and Cement Renderings, by C. Hobbs. College of Further Education, Warwick New Road, Leamington Spa. December 15. 7.15 p.m.

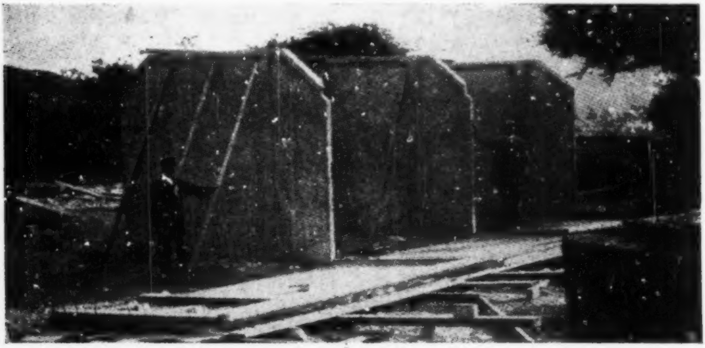
Practical Formwork Design and Construction, by J. G. Richardson. College of Further Education, Northgate Building, Darlington. December 16. 7.15 p.m.

Wire Ropes.

The dimensions and properties of wire ropes for use in civil engineering, mining, and other industries are given in a booklet issued gratis by the Tyne Wire Rope Manufacturing Co., Ltd., of South Shields. The data are arranged in four sections, namely ropes for mining and general engineering, ropes for lifts and elevators, ropes for excavators and other earth-moving machines, and non-standard ropes.

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", December, 1909.



Tilt-up Construction.—What is claimed to be the first use of the tilt-up method of construction in Great Britain is described in this number in connection with some farm buildings, one of which is illustrated, in Wales. It is stated that "The system employed of forming the sides and party walls flat on the ground and afterwards raising them and jointing the corners lends itself to cheap and quick construction. The method consists in moulding the slabs on platforms, which are temporarily hinged, then rearing the platforms with the slabs on them to a vertical position, lowering the platform for use with other sections, and finally securing the segments together."

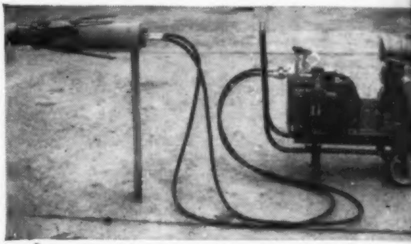
[A similar method was used for the construction of houses in the U.S.A. in 1906.]

PSC for al

FREYSSINET SYSTE

The illustration on the left shows the Freyssinet System of c
anchorage, with "Hydrarigid" sheathing. It is normally c
in the end of the member to be prestressed, and is usu
recessed to permit a protective mortar covering to be appl
after grouting. Anchorage cones are provided for 8/2
10/200, 12/200, and 12/276 cables.

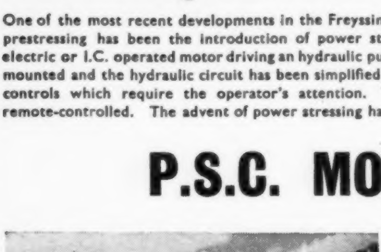
. . . CONE ANCHORAGE



FREYSSINET POWER EQUIPMENT . . .

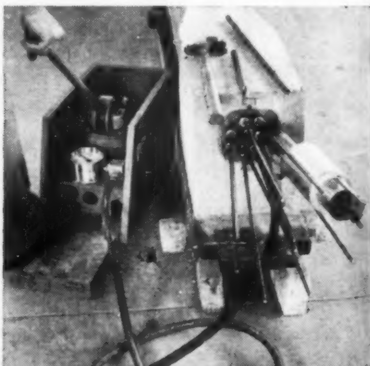
One of the most recent developments in the Freyssinet system of prestressing has been the introduction of power stressing. An electric or I.C. operated motor driving an hydraulic pump is trailer mounted and the hydraulic circuit has been simplified to only two controls which require the operator's attention. These are mounted on the trailer assembly and the jack is thus completely remote-controlled. The advent of power stressing has increased the rate of tensioning by three or four times that of hand pumping.

P.S.C. MONOWIRE SYSTEM



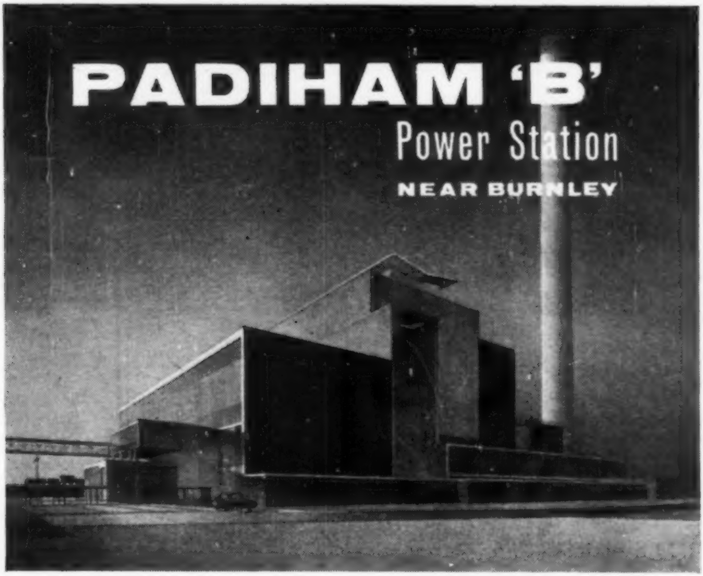
. . . P.S.C. ANCHORAGE

The 8/276 P.S.C. "MonoWire" Anchorage illustrated here shows a "Hydrarigid" welded seam-jointed metal sheath which screws into the anchorage. A new type of high impact plastic cable spacer is also available. These anchorages can be supplied for cables of one to twelve wires.



P.S.C. MONOWIRE POST-TENSIONING . . .

This illustration shows the P.S.C. lightweight single-wire jack stressing one of twelve wires on a 12/276 "MonoWire" anchorage. The jack is available with 4" or 6" extensions, and has a spring return.



From the ground up . . .

First the foundations, now the superstructure of the main power house and ancillary buildings. Gleeson have been entrusted with providing both base and backbone of the new Padiham ' B ' Power Station near Burnley.

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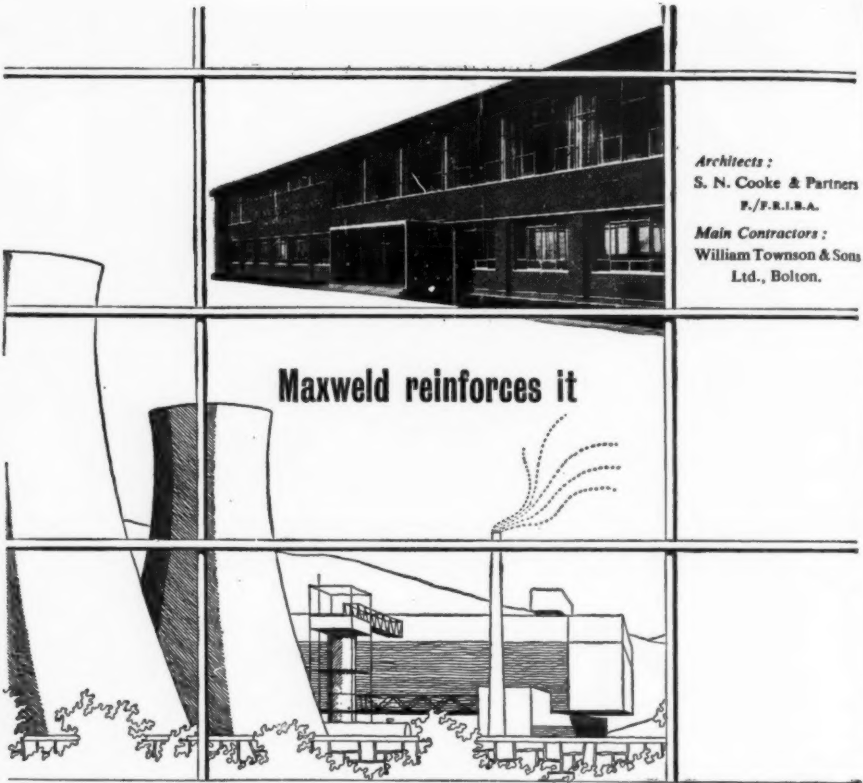
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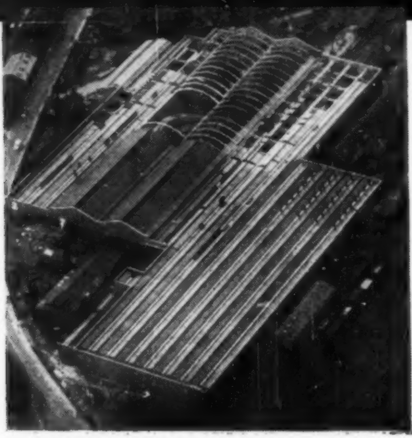
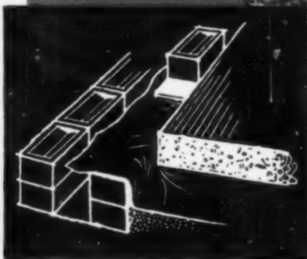


Leading contractors defy damp with 'Visqueen' oversite membranes



'Visqueen' 500 heavyweight, laid under the concrete ground floor rafts of a Wimpey House at Acklam, Middlesbrough. This estate is part of their national speculative house building programme.

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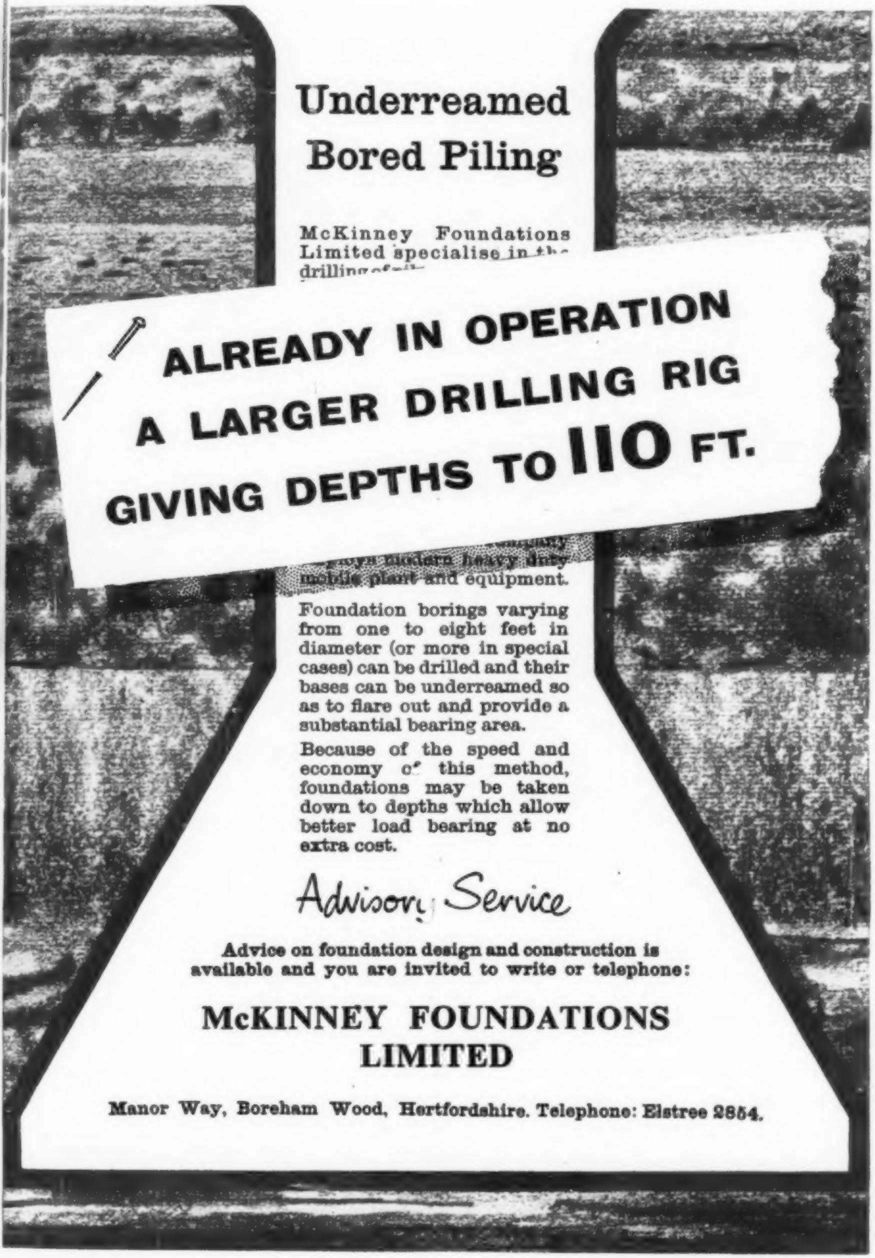
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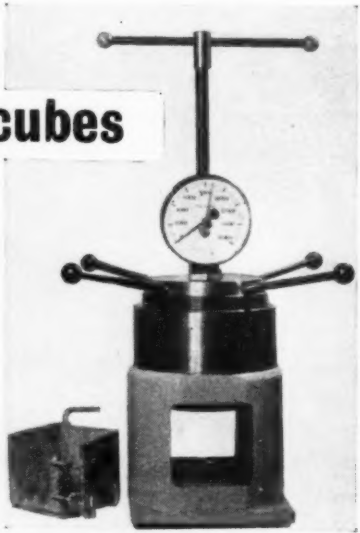

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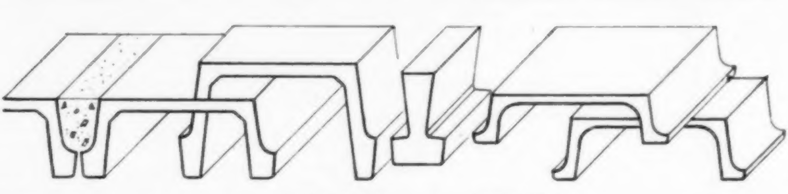

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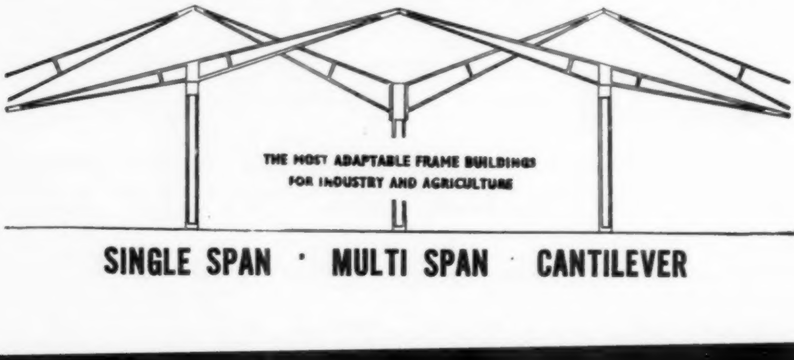

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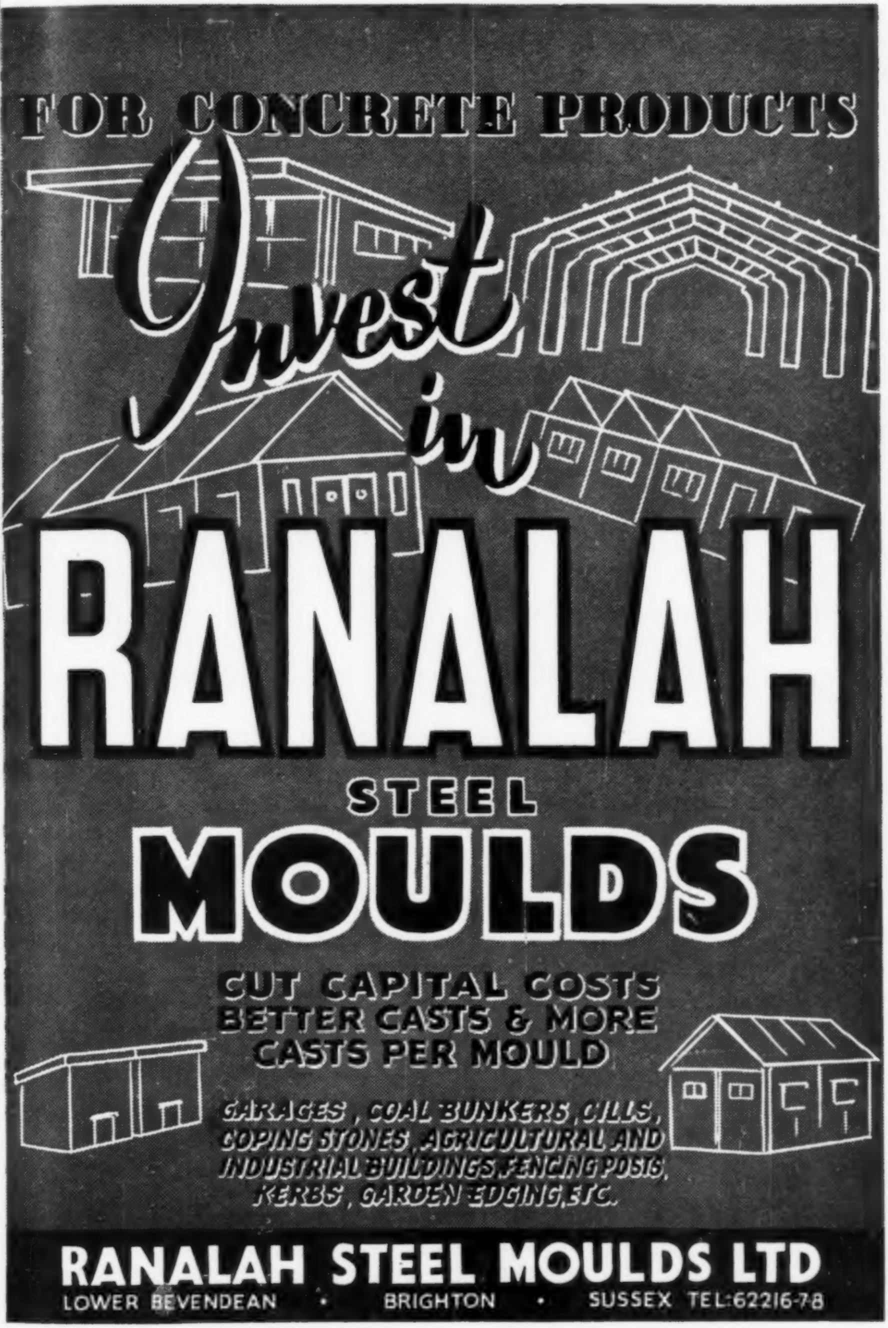
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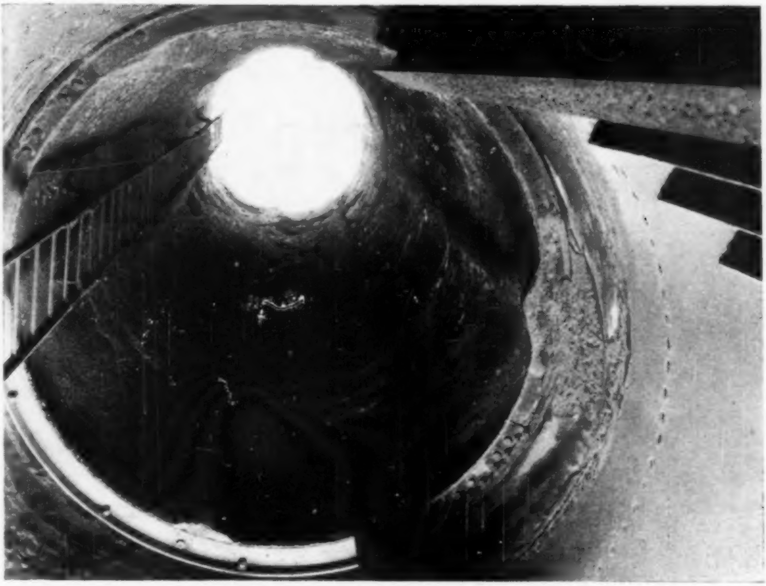

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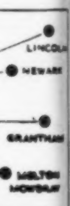

Published by CONCRETE PUBLICATIONS LTD., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
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